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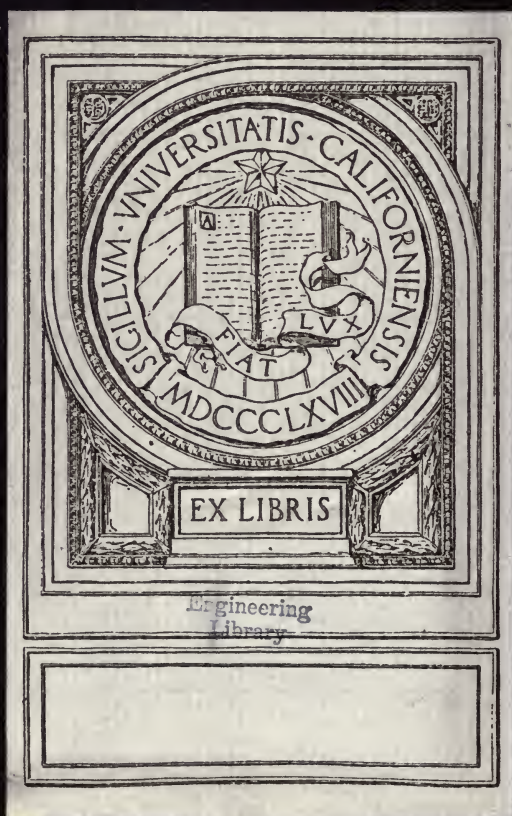
CIVIL ENGINEERING DEPARTMENT

UNIVERSITY OF CALIFORNIA

BY

PROFESSOR FRANK SOULÉ

1912





UNIVERSITY OF CALIFORNIA
DEPARTMENT OF CIVIL ENGINEERING
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Frank Louie



ORDINARY FOUNDATIONS.

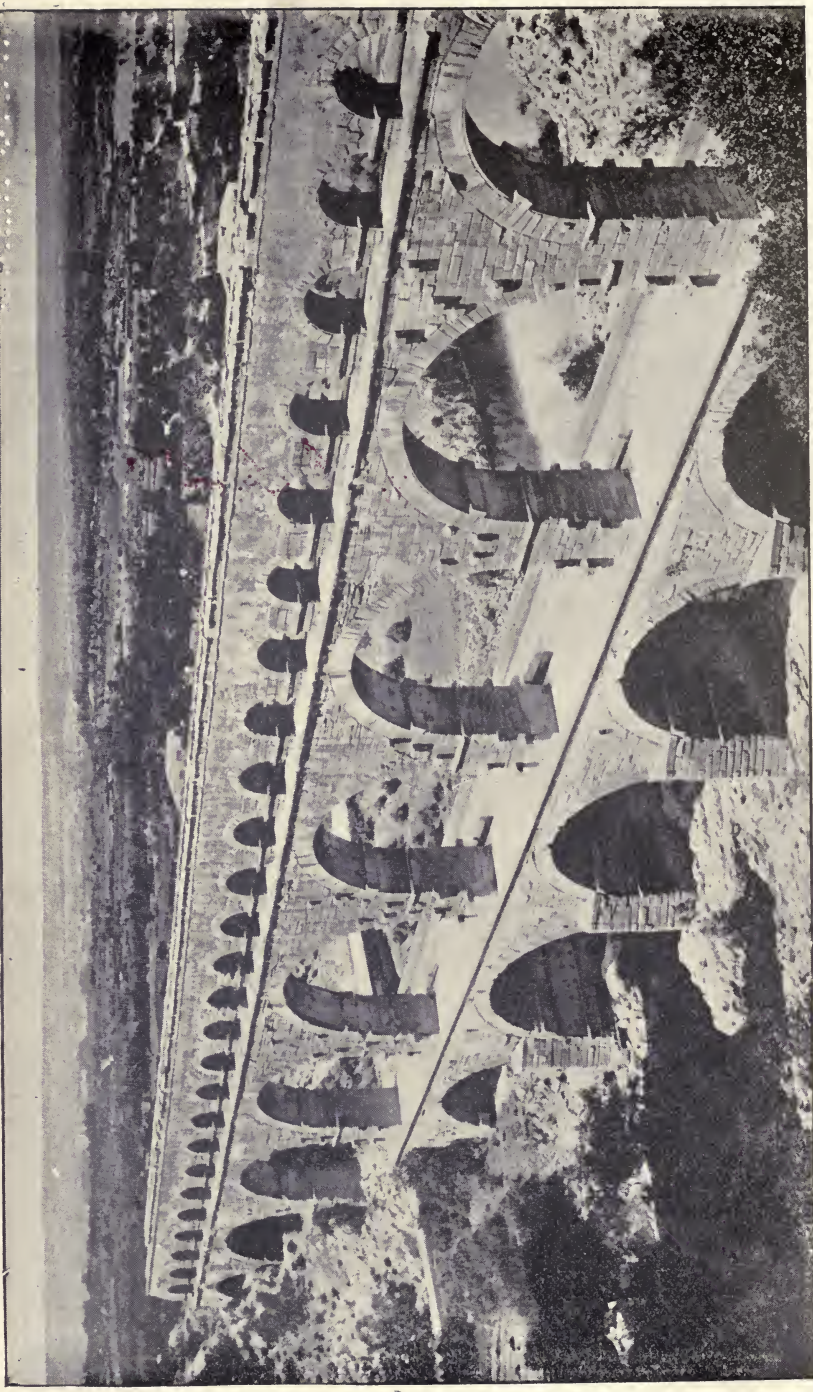
INCLUDING

THE COFFER-DAM PROCESS FOR PIERS.

FOWLER.



[Frontispiece.]



THE PONT DU GARD, NÎMES, FRANCE.

ORDINARY FOUNDATIONS.

INCLUDING

THE COFFER-DAM PROCESS FOR PIERS.

WITH

*NUMEROUS PRACTICAL EXAMPLES
FROM ACTUAL WORK.*



BY

CHARLES EVAN FOWLER,
CIVIL ENGINEER.

*Member American Society of Civil Engineers,
Member Society for the Promotion of Engineering Education,
President Pacific Northwest Society of Engineers,
President Seattle Board of Park Commissioners.*

"Much of the success of any one in any kind of work, and especially in work subject to the peculiar difficulties of that we are considering, depends upon the spirit in which it is undertaken."—ARTHUR MELLEN WELLINGTON.

SECOND EDITION, REVISED AND ENLARGED.

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BY

C. E. FOWLER.

PREFACE TO SECOND EDITION.

THE necessity for a second edition of this book has made it possible to make valuable additions to the text, by which the subject of ordinary foundations is more comprehensively covered. The construction of piers by the use of metal cylinders; with timber caissons by open dredging; and the construction of ordinary-sized foundations by the use of pneumatic caissons, has furnished another chapter. A chapter has been added on the subject of foundations, which covers the bearing capacity of soils.

The new chapter on building stone, masonry, and the design of piers is intended to supplement the old chapter on the "Location and Design of Piers."

Enough new matter has been added on cement and concrete to form an additional chapter, which includes valuable tables giving the amount of material required for concrete of different proportions.

The building of piers of timber and pile bents, together with the subject of timber preservation, has been discussed in a final chapter, as fully as a general knowledge requires.

It is hoped that in enlarging the field covered by "The Cofferdam Process for Piers," the book will be more valuable for the practicing engineer, besides giving a general enough treatment to make it valuable for class work.

C. E. F.

SEATTLE, 1904.

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INTRODUCTION.

THE greater part of foundation work is of an ordinary character. And while difficult foundations have been quite fully treated by engineering writers, ordinary ones have too often been passed over with mere mention, or treated in such a general way that the information proves of little value in actual practice.

Many valuable examples of work of this character have been described in current engineering literature, and it is hoped that by bringing them together a real service will be rendered the profession, as well as much valuable time be saved for considering other and equally important problems.

The history of the coffer-dam process would seem to indicate that engineers of nearly a century ago gave more consideration to the smaller problems than the engineer of to-day, who has apparently passed to the consideration of the larger and of course more interesting ones.

That this is deplorable, is proven by the many cases where money has been wasted in the after effort to make good the mistakes that have become apparent where cheap construction of coffer-dams has been resorted to. The saving in original cost, as between an indefensible method and a defensible one, is often so small as to seem absurd when it has become necessary to make large expenditures to rectify the errors.

Errors of judgment are more easily excusable with regard to foundations than with any other class of construction, but where definite limits can be set, economy will result by keeping as closely as possible within them.

Reference is made in the following pages to the splendid construction of foundations by the Romans, where they could be built outside the water. The Pont du Gard, illustrated in the frontispiece, is the most notable example of this extant. It is interesting also as indicating their knowledge of the better form of piers and methods of arch construction.

Although constructed during the reign of the Emperor Augustus, at the beginning of the Christian era, it is in a remarkable state of preservation, aside from repairs that have been made from time to time.

Probably the earliest recorded examples of the use of coffer-dams which give details of construction are those constructed under the engineers of the Ponts et Chaussées.

Those built under Perronet at the bridge of Orleans were large and extensive, and references made to the pile-drivers and the pumps used on the work, serve to illustrate the great amount of attention paid to planning the details of construction.

The same engineer completed the piers of the bridge at Mantes, where the coffer-dams were constructed to inclose both the abutment and the nearest pier within one dam, making the dimensions about 150 feet by 200 feet in the extreme.

Hardly less notable were the coffer-dams at Neuilly, where the interiors were so large that the excavation did not approach near the inside wall of the dam.

All of these were constructed prior to the year 1775, and the details as shown in the elaborate drawings are of much interest to the engineer engaged on similar works.

The coffer-dams constructed about 1825 by Rennie on the new London bridge were the prototypes of those used at Buda Pesth, but were elliptical in form. They were designed with as much care, apparently, as any other feature of the bridge, and from the fact that the water was pumped to twenty-nine feet below low water and the work found tight, the details must have been very carefully executed.

However great the amount of care bestowed, there will be cases undoubtedly where the difficulties cannot be foreseen, and

it will become necessary to adopt some of the many expedients cited to overcome them; or they might better be employed from the start, where any suspicion is had that trouble may ensue.

The question as to whether it will be best to use a crib or a sheet-pile coffer-dam will most always be decided by the character of the bottom, the location, and the character of the foundation to be built. It is advisable, whichever type is selected, to make the size large enough, so that the excavation may be completed without approaching too close to the inside wall of the dam, and so that plenty of room may be had for the laying of the foundation-courses.

The unit stress adopted for timber construction is believed to be as large as will give good results in the majority of cases, both on account of the possibility of the construction having to undergo more severe usage than is expected, and on account of the grade of timber which is most often made use of for temporary works.

Where it is permissible from the standpoint of true economy, it is believed that steel construction will commend itself for use. In most localities it will not be long until metal construction will be found cheaper than timber for building coffer-dams, and in many places this is already true.

A great mistake is made, in nearly nine cases out of ten, by trying to use old machinery, such as hoisting-engines, pumps, and the like, which are ill adapted to the purposes for which they are intended, on account of lack of capacity, and only too often on account of having outgrown their usefulness.

The engineer would avoid many unpleasant situations by demanding that a proper outfit be provided, and in the end gain the thanks of the contractor for increased profits.

Extended acquaintance with Portland cement is increasing the use of concrete in construction, and this is a great gain for the engineer, as it is not only superior to much stone that is used, but is better adapted to use in difficult situations. It also lends itself more readily to use for ornamental details in pier construction. That truly ornamental piers are not, however, those with

needless and frivolous details, has been clearly set forth in the last article. Simplicity and beauty are near relatives.

The best locations cannot always be chosen for piers, but careful examination will often be the means by which bad locations may be avoided.

The methods for determining the economic division of a given crossing of a river have not come into general use, probably on account of lack of easy application. The method given is an accurate one and very simple to use, especially if the results are tabulated for a given loading.

C. E. F.

NEW YORK CITY, 1900.

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TABLE OF COFFER-DAMS.—

No.	Page.	River and Location.	Current.	Water Head.	Character of Bottom.
1	6	River 200 feet wide, Ohio.....	None.	12'+	Cemented gravel.
2	7	Clyde at Glasgow.....	Slight.	9'+	Gravel, sand, mud.
3	9	Estuary or Harbor.....	Tide.	40'	Sand & gravel over clay.
4	10	Danube at Buda Pesth.....	Swift.	54'±	Gravel over clay.
5	17	Kanawha near mouth.....	Swift.	34'—	Gravel over hard-pan.
6	18	Ohio near head.....	Moderate.	20'+	Gravel.
7	19	Western part United States...	Moderate.	6'+	Soft.
8	20	St. Lawrence, lower river....	Swift.	20'+	Rock.
9	20	Arnprior Bridge.....	Swift.	21'+	Rock.
10	23	New Mexico, underflow.....	None.	15'+	Sand.
11	23	Arkansas at Tulsa.....	Moderate.	7'+	Gravel over rock.
12	23	Western part United States...	Moderate.	6'+	Soft.
13	23	Republican in Kansas.....	Moderate.	6'+	Sandy.
14	23	Western part United States...	Moderate.	7'+	Gravel over soapstone.
15	23	Western part United States...	Moderate.	6'+	Rock.
16	23	Payette and Weiser, Union Pac.	Moderate.	6'	Soft.
17	25	Mississippi, Fort Madison....	Swift.	19'	Soft.
18	25	Schuylkill near Phila., Pa....	Moderate.	Deep.	Mud over rock.
19	35	U. S. Canal, Keokuk.....	None.	12'+	Rock.
20	39	Mississippi, St. Louis.....	Swift.	22'	Rock.
21	40	Queen's Bridge.....	Swift.	15'	Rock.
22	43	Harlem Ship Canal.....	Moderate.	25'	Rock.
23	43	Arthur Kill Bridge.....	Tide.	28'	Clay over rock.
24	45	Coteau Bridge, C. Pac. Ry....	Moderate.	28'	Rock.
25	68	Ann Arbor, Mich., M. C. Ry..	Moderate.	6'+	Gravel.
26	68	Arthur Kill Bridge.....	Tide.	30'—	Mud and clay.
27	70	Boston Harbor, sewer.....	Tide.	10'	Sand and gravel.
28	73	Illinois River, La Grange....	Moderate.	7'	Sand and mud.
29	73	Kankakee at Momence.....	Moderate.	6'+	Rock.
30	74	Potomac at Harper's Ferry...	Swift.	6'+	Rock.
31	74	Charlestown Bridge, Boston...	Tide.	6'+	Soft.
32	74	Fort Monroe, sewer.....	None.	20'	Soft.
33	78	Tennessee at Chattanooga...	Swift.	8'+	Gravel over rock.
34	79	Cumberland, Md.....	Moderate.	10'+	Sand over hard-pan.
35	80	Mississippi, Sandy Lake.....	Swift.	8'+	Sand.
36	83	Arkansas, Little Rock.....	Moderate.	6'+	Sand.
37	84	Parnitz, Stettin, Germany....	Moderate.	25'+	Clay.
38	87	Coosa, Gadsden, Ala.....	Moderate.	10'+	Gravel over rock.
39	88	Schuylkill, P. & R. R.R.....	Swift.	8'+	Rock.
40	91	St. Helier Bridge, Jersey, Eng.	Tide.	13'+	Earth over rock.
41	92	Thames at Putney.....	Moderate.	Deep.	Mud.
42	92	Victoria (B. C.) Docks.....	Tide.	35'	Rock.
43	92	Kaw at Topeka.....	Swift.	6'+	Sand.
44	101	Firth of Forth.....	Tide.	15'+	Rock.

SYNOPSIS OF EXAMPLES.

Form of Construction.	Inside Dimensions.	Kind of Puddle	Thick-ness Puddle.	Remarks.	Page.	No.
Earth bank.	10' × 60' ?	Clay and gravel.	5' +	No leaks.	6	1
Sheet-piles.	20' × 58' ?	Clay.	3'		7	2
Sheet-piles.	Large.	Clay, sand & gravel.	3-6'	Typical.	9	3
Sheet-piles.	72' × 136' +	Clay and gravel.	2-5'	Difficult.	10	4
Earth bank.	90' × 330'	Clay and gravel.	19' +		17	5
Earth bank. ?	200' × 600'	Clay and gravel.		Failed.	18	6
Crib.	Medium.	Clay.	3' +		19	7
Crib, single.	24' × 43'	Concrete inside.			20	8
Crib, single.	16' × 34'	Concrete inside.			20	9
Crib, single.	17' × 43'	Clay outside.		Special.	23	10
Crib, single.	Medium.	Clay outside.			23	11
Sheet-piles.	Medium.			Typical.	23	12
Sheet-piles.	Medium.	Clay outside.			23	13
Sheet-piles.	Medium.	Clay outside.			23	14
Sheet-piles.	Medium.	Clay.	} Equal depth.		23	15
Box or crib.	12' × 36'	None.		On grillage.	23	16
Staves.	36' diam.	None.		On grillage.	25	17
Sheet-piles.	80' diam.	None.		Failed.	25	18
Canvas on plank.	80' long.	Rotten manure.		Bulkhead.	35	19
Crib, double.	28' × 64'	Clay.	3' 0''	Canvas used.	39	20
Box and canvas.	Square.	Clay outside.		Movable.	40	21
Polygon crib.	47' diam.	Clay.	4' 6''		43	22
Polygon crib.	44' diam.	Clay and gravel.	5' 0''		43	23
Crib, single.	34' diam.	Concrete inside.			45	24
Sheet-piles.	13' × 44'	Clay and gravel.	2' 8''		68	25
Sheet-piles.	Large.	None.		Two trials.	68	26
Sheet-piles.	12' wide.	Clay.	6-8'		70	27
Sheet-piles.	Medium.	None.			73	28
Sheet-piles.	Medium.	Gravel.		Two trials.	73	29
Sheet-piles.	Medium.	Gravelly clay.			74	30
Sheet-piles.	18' 6'' × 119'	Concrete inside.			74	31
Sheet-piles.	44' diam.	Sand and concrete.	7' +		74	32
Sheet-piles.	Large.	Clay.	9' 0''		78	33
Sheet-piles.	15' × 50'	None.			79	34
Sheet-piles.	829' long.	Clay.	8' ±		80	35
Sheet-piles.	16' × 38'	Earth outside.			83	36
Sheet-piles.	23' × 55' ±	Clay.	2-4'	Removal.	84	37
Sheet-piles.	28' × 28' ±	Clay.	12' +	Removal.	87	38
Sheet-piles.	16' × 42'	Clay and gravel.	8' +	Movable.	88	39
Sheet-piles.	Medium.	Clay outside.			91	40
Sheet-piles.	Medium.	None.			92	41
Sheet-piles.	500' long.	Clay.	2-7'		92	42
Sheet-piles.	18' × 55'	Clay outside.			92	43
Metal.	60' diam.	Concrete seal.			101	44



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ORDINARY FOUNDATIONS.

CHAPTER I.

HISTORICAL DEVELOPMENT.

THE continued increase in the weight of our bridge superstructures and of the loads they have to carry has led to increased care, to a very gratifying degree, in the preparation of the foundations for bridge piers and abutments.

An old authority very truly states "The most refined elegance of taste as applied in the architectural embellishment of the structure; the most scientific arrangement of the spans and disposition generally of the superior parts of the work; and the most judicious and workmanlike selection and subsequent combination of the whole materials composing the edifice, are evidently secondary to the grand object of producing certain firm and solid bases whereon to carry up to any required height the various pedestals of support for the spans of the bridge."

There is every reason to believe, from the bridges of the Romans still extant and of those of ancient and mediæval times of which there are remains or records, that the foundations were carefully considered.

The most ancient form was likely begun by dumping in loose stones until the surface of the water was reached and the masonry could then be commenced without the necessity for any method of excluding the water. The oldest civilizations were in tropical or semi-tropical countries where the streams are dry beds for

many months in the year and suitable foundations were easily made without water to contend with. Where the bottom of the stream was rock, the engineering could be very little improved upon to-day, and even where there was shallow water on rock bottom, the piers were well founded in the shallowest places, the bridge often winding across the stream in serpentine form, similar to the bridge over the river Karun, at Shuster, Persia. (Fig. 1.)

The arch was developed to such an extent by the Romans, and the spans were increased to a length which rendered the construction of piers in the water unnecessary for short bridges, the abutments or skew-backs being without the stream on either bank.

The difficulty of founding piers in midstream was doubtless the controlling cause for the larger spans, such as the one built at Trezzo, over the river Adda, by order of the Duke of Milan, some time prior to the year 1390. The span at low water was 251 feet, the single arch being of granite in two courses. The placing of a middle support was doubtless found to be impracticable and caused the design of an arch which has never been equaled or eclipsed. (Fig. 2.)

The construction of roads has ever been the harbinger of civilization, and with the spread of civilization came a demand for the improvement of means of communication. The engineer was called upon to construct better and greater bridges in a permanent manner, which led to the origin and development of the four methods for founding in water that were used in olden times. These may be classified as, first, the method with open caissons; second, the use of piles with a capping of coarse concrete about the tops; third, the use of piles after the manner of the French encaissement; and fourth, the use of coffer-dams. A fifth method might be added, in which the bridge was built on dry land adjacent to the stream, and the river diverted to a new channel afterwards excavated under the completed structure. This is, however, an avoidance rather than a solution, unless the river is to be diverted in the course of its improvement.

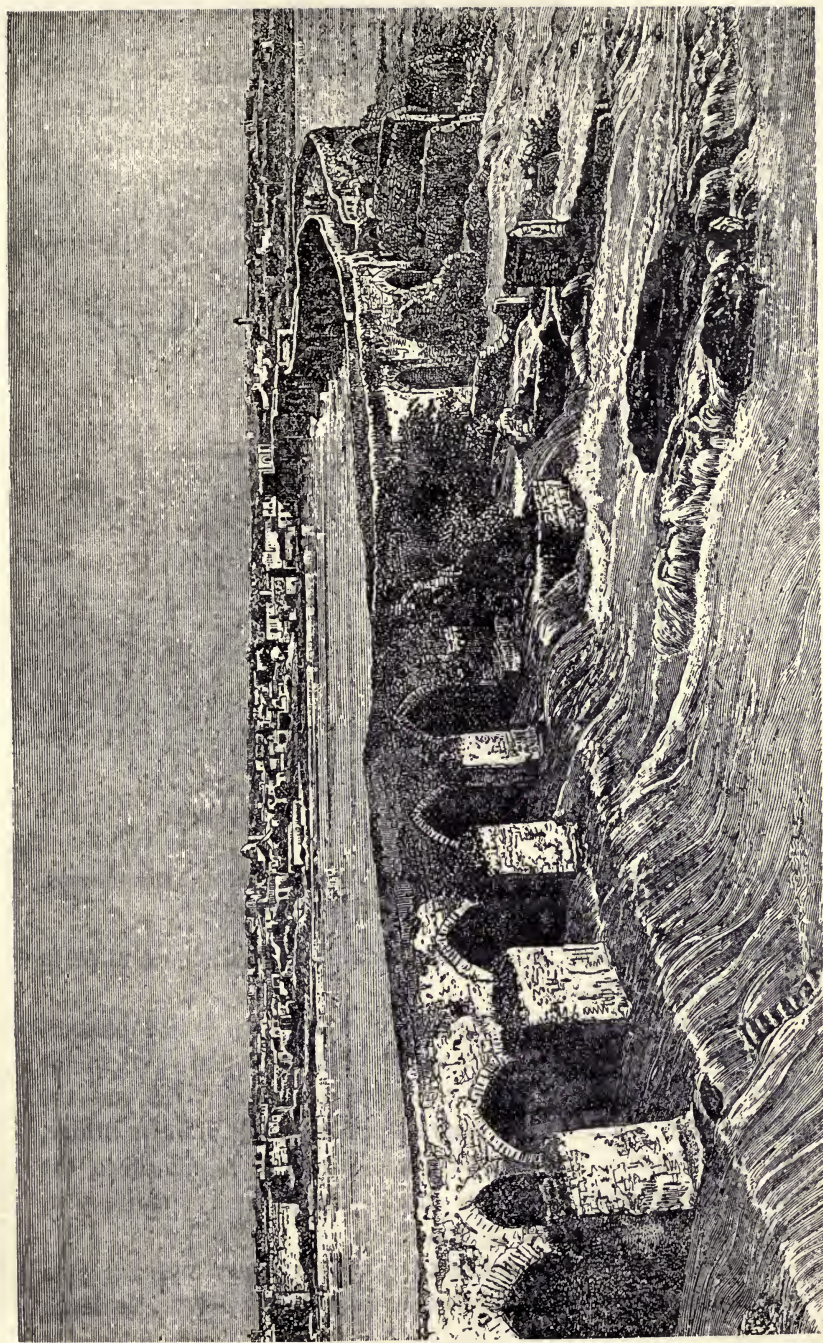


FIG. 1.—BRIDGE AT SHUSTER, PERSIA, OVER THE RIVER KARUN.

The first method, as described in old treatises or accounts, consisted of little more than baskets formed of branches of trees, weighted with stone to sink them, and after sinking filled with loose stone to near low-water level, where the masonry could be commenced. These baskets were similar in construction to the mattresses used in the bank revetment of the Mississippi or

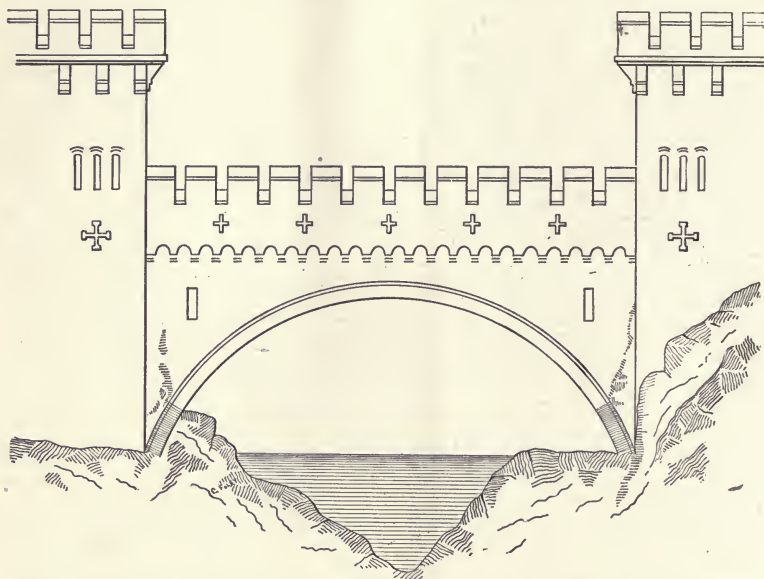


FIG. 2.—BRIDGE OVER THE ADDA AT TREZZO, MILAN, A PROBABLE RESTORATION.
(The shaded portion of arch rings is all that remains.)

the bamboo casings used by the Japanese to hold stones in place on bank protection.

An improvement was effected by using in place of baskets, boxes or small open caissons which were sunk and filled in the same manner, several being used for one pier. This was the method used at Blackfriars bridge and also at Westminster bridge, over the Thames, and has been much used in recent times, the caisson being built large and strong enough for the entire pier, which is built up as the caisson sinks.

The second method consisted of driving piles over the area of the foundation until the heads were below low-water level,

and spaced at distances apart as required by the nature of the bottom, similar to the methods in vogue to-day. The heads of the piles were not driven to the same level, however, and were incased in a form of coarse concrete such as was used by the Romans, but what is now called beton. This was leveled up and on it was laid the stone for the footing course of the pier.

The third method of encaissement was probably an improvement of the dumping in of loose stone on which to place the pier, and consisted in inclosing the space for the pier with sheet-piling, after which the loose material was removed from the bottom as

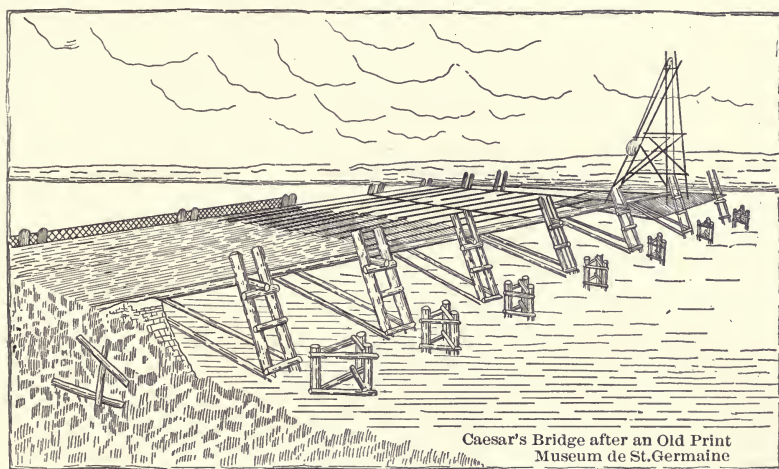


FIG. 3.—CÆSAR'S BRIDGE OVER THE RHINE.

much as possible and the stone dumped inside until nearly up to low water, at which time the pier could be begun.

These last two methods doubtless met with much favor, owing to the familiarity with pile-driving, in which the Romans especially were proficient. Cæsar's bridge over the Rhine was built entirely on piles, and in a view of it after the old print in the Museum de St. Germaine, is pictured a pile-driver on a float in position for driving. (Fig. 3.)

This third method was the early type of the crib, which has been such a factor in the building of the earlier foundations over our American rivers. Crossed timbers laid up crib fashion

with rectangular openings or cells between the timbers were sunk and filled with broken stone on which to build the pier.

These methods were all deficient in affording no means of seeing or making a careful examination of the bottom on which the foundation was to be placed, and with the advent of more permanent structures of greater magnitude the coffer-dam came into use. This allowed the bottom to be freed from water, and after a careful examination and preparation of the foundation, the work could proceed in the dry until above water-level.

The pneumatic caisson is now in general use for all foundations that must go to any considerable depth below the water and has even been used in some instances where the depth was slight, but where for various reasons it was deemed expedient to use compressed-air caissons. Recent expressions from some engineers of high standing would indicate that they do not consider it good practice to use coffer-dams in any case, one making the statement that he had not used a coffer-dam for thirty years, while another seemed to think it a matter to be left to the pleasure of the contractor. That the use of this method has gotten into disfavor seems to be beyond question and it will be the purpose of the succeeding pages to learn to some extent why this is so, but mainly to show from successful examples how to proceed, that success instead of failure may result. Any attempt to account for the origin of the coffer-dam process would be futile, inasmuch as the savage, wishing to free a space from water, doubtless banked up earth about the area and, scooping out the water with his hands, laid the ground bare for inspection. From so simple a beginning, the first method likely to occur to a mind capable of reasoning, can readily be imagined the course of development of coffer-dams.

The most simple form in use at the present time, where the water is quiet, is shown in Fig. 4, and consists principally of a bank of earth which is made thick enough to be nearly or quite impervious to water, the earth being prevented from caving into the excavation by piles supporting a timber casing. Some of the recorded examples of the early use of this process

are of interest in illustrating the care which has bestowed upon their construction in important works and will call attention to that incessant care which is necessary to success in any work of this character.



FIG. 4.—A PRIMITIVE SOLUTION.

Robert Stevenson, the great English engineer, thought it not beneath his dignity to give full instructions as to the construction of the coffer-dams for the Hutcheson bridge over the Clyde at Glasgow. The bridge consisted of five arch spans, the total

length between the abutments being 404 feet and the width 38 feet. The four piers were from 11 to 12 feet in thickness, being designed to take up the arch-thrust, and 48 feet in length at the footing. The specifications written at Edinburgh in April, 1828, are so explicit that they will be quoted in full on this point: "It having been ascertained by boring and mining that the subsoils of the bed of the river consist of gravel, sand, and mud to the depth of 27 feet and upwards, it becomes necessary to prepare foundations of pile-work for the bridge; and, therefore, to insure the proper and safe execution of the works, coffer-dams are to be constructed around each of the foundation-pits of the two abutments and four piers of such dimensions as to afford ample space for driving piles, fixing wale-pieces, laying platforms, pumping water, and setting the masonry; and likewise for the construction of an inner or double coffer-dam should this ultimately be found necessary. The framework of the coffer-dams is to consist of not less than two rows of standard or gage- and sheeting-piles, kept at not less than 3 feet apart for the thickness of a puddle-wall or dyke, which space is to be dredged to a depth of not less than 9 feet under the level of the summer water-mark above described, before the sheeting-piles are driven. The gage or standard piles are to measure not less than 24 feet in length and 10 inches square. They are to be placed 3 yards apart and driven perpendicularly into the bed of the river to the depth of 16 feet under the level of the summer water-mark, thereby leaving 8 feet of their length above that mark. Runners or wale-pieces of timber 9 inches square are then to be fitted on both sides of each row of gage-piles, to which they are to be fixed with two screw-bolts of not less than 1 inch in diameter, passing through each of the gage-piles. One set of these inside and outside wale-pieces is to be placed at or below the level of summer water-mark, and the other set within 1 foot of the top of each row of said piles, the whole to be fixed with screw-bolts in the manner above described. The wale-pieces are to be $4\frac{1}{2}$ inches apart in order to receive and guide the sheeting-piles. This is to be effected by notching the wale-pieces into the gage-

piles. The sheeting-piles are to be 21 feet in length, $4\frac{1}{2}$ inches in thickness, and not exceeding 9 inches in breadth. They are to be closely driven, edge to edge, along the space left between the wallings, and each compartment of the sheeting between the gage-piles is to be tightened with a key-pile. The coffer-dam frames are to be properly connected with stretchers and braces before commencing the interior excavation. Each coffer-dam is to be provided with a draw-sluice, 14 inches square in the void, with a corresponding conduit passing through the puddle-dyke at the level of summer water-mark. To render the coffer-dams water-tight the whole excavated space between the two rows of piling is to be carefully cleared of gravel, sand, or other matters, to the specified depth, and clay well punned or puddled is then to be filled in and carried up to the level of the top of the sheeting-piles. But if it shall, notwithstanding, be found that the single tiers of coffer-dam do not keep the foundation-pits sufficiently free of water for building operations, the water must either be pumped out and kept perfectly under by steam or other power, or else excluded by the construction of a second tier of coffer-dam similar in construction to the first. For the foundation-pits of the two abutment piers on either side of the river it is not expected that more will be required on the landward side for keeping up the stuff than a single row of gage- and sheeting-piles; but if the engineer shall find other works necessary upon opening the ground they must be executed by the contractor and shall be paid for agreeably to the contract schedule of prices for the regulation of extra and short works. The stuff within the coffer-dams is to be excavated to the depth of 10 feet under the level of summer water-mark for each of the piers and 8 feet for each of the abutments."

The present practice of leaving all this to a contractor, whose idea is too often to sacrifice everything to cheapness, appears in very unfavorable contrast to this careful description.

An article on founding by means of coffer-dams, published in 1843, gives directions for placing a coffer-dam in 40 feet of tide-water; and although the engineer of to-day would use some

other method for such a depth, an illustration (Fig. 5) and short description of it are given, as ideas may be gained for application to ordinary works.

The water was assumed at 10 feet deep for low tide, 28 feet at high tide, with 12 feet of sand and gravel to be removed to expose the clay on which the pier was to rest. Four rows of piles were to be driven around the area, the outer row to within 1 foot of low water, the two rows in the middle to within 3 feet of high water, the inner row to 11 feet above low water, and all

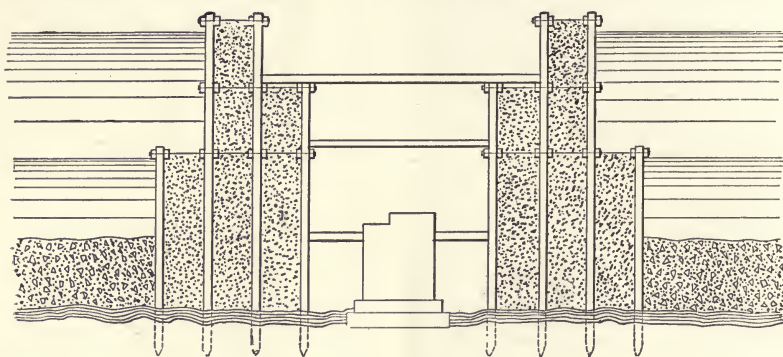


FIG. 5.—COFFER-DAM IN TIDE-WATER.

to be down 5 feet into the clay. The outer row of piles to be $6'' \times 12''$, the two rows in the middle $12'' \times 12''$, and the inner row $8'' \times 12''$; all driven close together and to have waling-pieces, braces, and brace rods as shown in cross-section. The rows to be 6 feet apart and to be filled in between with a puddle of clay mixed with sand and gravel.

The report of W. Tierney Clark, the engineer of the Buda Pesth suspension bridge, gives an account of what are probably the largest bridge coffer-dams ever constructed. Some other method would now be used for such a location, but this fact will not detract from the lessons that may be drawn from them.

The Danube was crossed at Buda Pesth previous to the year 1837 by means of a bridge of boats which had to be taken up during the winter and the passage made by ferry or on the ice, so that for six months of each year there was great risk in crossing

and frequent loss of life. The building of a permanent bridge was brought about through the efforts of the Count Szechenyi, who, as a member of a committee, proceeded to England in 1832 and after a careful investigation of existing works decided upon the construction of a suspension bridge. The greatest question for solution was the founding of the two towers in a river like the Danube, where the ice throughout the long winter wrought havoc with everything in reach. The ice in the river in February, 1838, was from 6 to 10 feet thick near the site of the proposed

TRANSVERSE SECTION NO.3 COFFER DAM

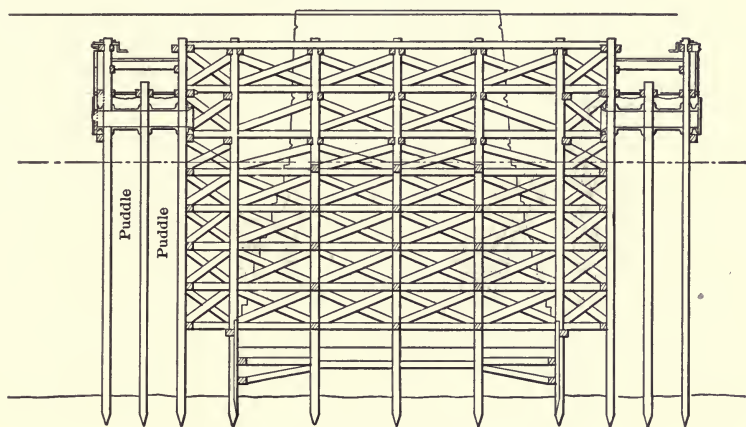


FIG. 6.—BUDA PESTH SUSPENSION BRIDGE.

piers. On March 9 a movement occurred across the whole river and for a length of 350 yards, the whole moving in a solid mass. On March 13 it moved again 400 yards and three hours later a general breaking began. The ice piled up on the shoals, causing a sudden rise to 29 feet 5 inches above zero, and while it was at this height for only a few hours, it is recorded that a great part of Buda and two-thirds of Pesth were destroyed and many lives lost.

The extraordinary design of the coffer-dams can the more readily be understood after this description, it being doubted by many persons at that time whether piers could be placed in the river by any means. (Fig. 6.)

The drawings reproduced are of coffer-dam No. 3, which was about 72 feet in width and about 136 feet in length inside the puddle walls, there being two puddle chambers, each 5 feet in width. From a point about 13 feet above the clay on which the tower was to rest, was an inside wall of sheet-piling, this space being nearly filled, after excavating, with a bed of concrete. The piling of each row, from 40 to 80 feet in length, was all carefully sized to 15 inches square, shod with iron and driven close together, penetrating 20 feet below the bed of the stream or 40 feet below the zero level. The framing of the ice-breaker and the bracing within the dam was of enormous strength. The number of piles driven in the four coffer-dams reached the enormous total of 5,224, and of the 1,227 driven in dam No. 3, 16½ per cent. were drawn and redriven. These piles and the timber were obtained from the forests of Bavaria and Upper Austria. (Fig. 7.)

The first pile on dam No. 3 was set on April 8, 1842, but owing to the difficulties encountered it was not finished until three years later—April 4, 1845. From six to seven days were occupied at the first in driving a pile to a depth of 5 or 6 feet into the clay, but as the work progressed the difficulty increased, the operation of driving one pile consuming from twelve to fourteen days, many piles breaking short off so they could not be withdrawn, and the gravel was dredged out from behind and a second row driven. The report further describes the difficulty of the work: "The dredging for the No. 3 dam was carried on to the average depth of 44 feet from the top of the outer row of piles, leaving about 10 feet of gravel to drive through, and extra piles were driven where the gravel found its way between the piles, as well as where it was known the piles were not driven to the proper depth, or were broken or otherwise injured. As the gravel was dredged out to the above depth, the inner and middle row of piles were driven, and a great part of them got down as was supposed to the requisite depth. The work was carried on in the above manner until the 7th of November, when from the appearance of several piles which were pulled up, and from other causes, it became apparent that the outer row was in

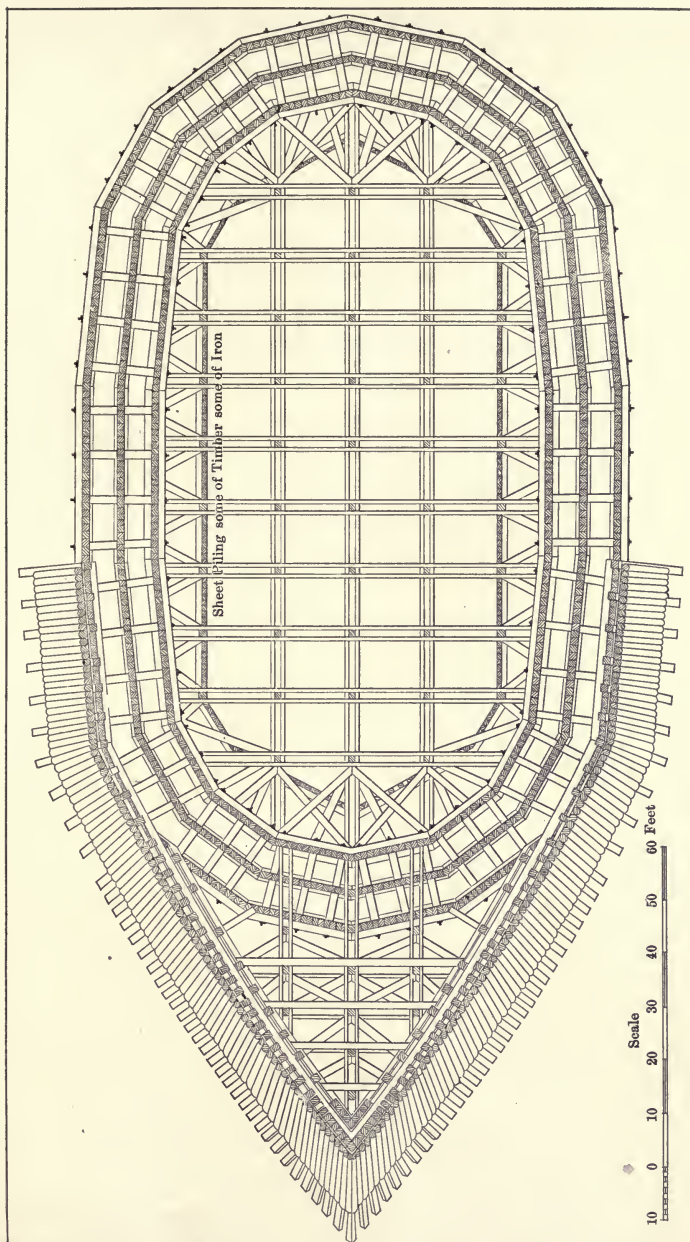


FIG. 7.—BUDA PESTH SUSPENSION BRIDGE, PLAN OF COFFER-DAM No. 3.

a much worse state than had been expected, and it was almost a matter of certainty that those piles which had taken ten or twelve days to get down were not driven to the proper depth by at least 3 or 4 feet, having upset or lost their points to that extent. There was likewise every reason to believe that many of them were broken or dangerously crippled. Added to this the Danube was rising, and at the late time of the year, with winter rapidly approaching, the general appearance of the dam was anything but satisfactory. Upon mature consideration the only course appeared to be to drive a much greater number of piles than was at first calculated upon, and another complete row of piles was driven all round at intervals of 15 inches apart, and in some cases double and triple piles were driven during the progress of the dredging. At the commencement of the driving a few were got down to the depth of 57 or 58 feet, being from 3 to 4 feet in the clay; but as the gravel began to get compressed many of them would not penetrate more than 54 or 55 feet, the sharp, angular gravel overlying the clay appearing to be compressed into a substance as hard as rock."

The puddle used was clay mixed with about one-third clean gravel, it having been found to set quite solid, from experiments made by sinking specimens in the Danube. When leaks occurred they were closed by driving square timbers down 30 or 40 feet into the puddle to pack it, or by driving new piles to close the cracks, and in some cases by driving sheet-piling.

Experiences of this nature led to the disuse of coffer-dams for foundations to such depths, but a very small percentage of the care exercised and the persistence shown in this work would lead to greater success on ordinary foundations.

The class of work to which coffer-dams may still be applied will be shown in the succeeding pages and the examples from actual practice will show in some measure the care that must be exercised in the first construction to prevent failure, and the expedients adopted to overcome unavoidable accidents.

"In every man's mind, some images, words, and facts remain, without effort on his part to imprint them, which others forget, and afterwards these illustrate to him important laws."

CHAPTER II.

CONSTRUCTION AND PRACTICE.—CRIB COFFER-DAMS.

THE exact definition of the term coffer-dam—"a water-tight inclosure, from which the water is pumped to expose the bottom and permit the laying of foundations"—is the class of structure which is to be considered, although in the construction of them cribs or caissons may be employed and utilized; the essential purpose being to form an inclosure as nearly water-tight as possible in order that the expenditure of power for pumping out the water may be of small amount.

The attainment of this when the water is shallow and has little current we have seen to be easily accomplished by means of a bank of clay or clayey gravel.

This form may also be employed in still water up to about 4 feet in depth by the addition of sheet-piling or a casing supported by ordinary piles to prevent the embankment from caving into the excavation. Where the bottom is of soft mud or porous material over a solid clay or gravel, as much as possible of the porous material should be removed before forming the embankment, thus preventing leakage underneath. In very shallow water this can be accomplished by shoveling and with large hoes or scoops, but with several feet of water to contend with, some form of dredge or scraper must be employed. A very convenient form of scraper used by M. L. Byers on the Cinti. & Mus. Valley Railway is described in Vol. 31 of the "Transactions of the American Society of Civil Engineers," and consists of old boiler-iron, strengthened by three ribs of light iron rail, as shown in Fig. 8. This was operated by a double-drum 20-horse-

power Mundy hoisting-engine, with the towing-line running directly from one drum to the scraper and the back line from the other drum over a sheave to the front of the scraper. The excavating averaged about 45 yards of material each day during twelve days' work. The weight of the device was about one thousand pounds.

Where the material is very soft, a hand-dredge, called a spoon, will accomplish the work at about the same cost as excavating

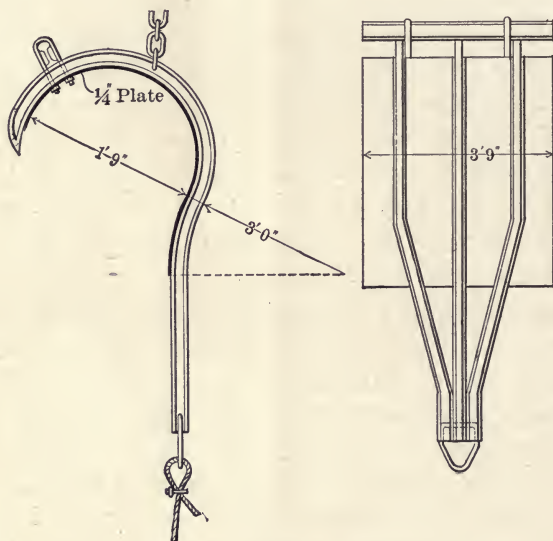


FIG. 8.—SCRAPER DREDGE.

on dry land. The spoon usually consists of a long pole, having a cutting-ring fastened at one end, and to this ring is attached a canvas bag to contain the excavated material. The ring is hung from a derrick with a set of falls, being guided with the pole, as it is dragged forward by the derrick through the material to be excavated.

Excavating may be done on all the larger rivers by employing the sand or gravel diggers which are almost always to be found, the dredging being accomplished by means of a series of buckets on a belt or on chains operated through a well in the bottom of

a barge. Dredging by machinery on a large scale will be considered later on in some detail.

The method of embankment is sometimes employed for greater depths than 4 feet and in some instances successfully.

The Chanoine dams on the Great Kanawha River required

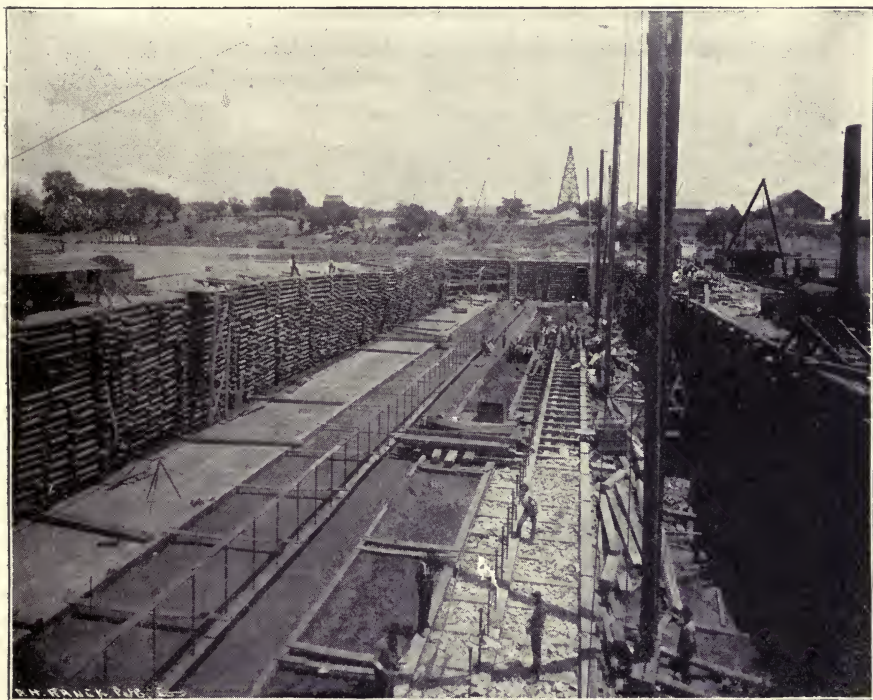


FIG. 9.—COFFER-DAM AT DAM NO. 11, GREAT KANAWHA RIVER.

substantial foundations beneath the water, and to accomplish this Addison M. Scott, the resident engineer, employed log cribs about the spaces, with earth banked up on the outside. This work is described in the report of the Chief of Engineers for 1896.

The site of the navigation pass of dam No. 11, including the center pier, required a coffer-dam 90 feet wide and 330 feet long inside. (Fig. 9.) This area, including the necessary room for the cribs, was dredged out to hard-pan from 20 to 24 feet below low water. The long cribs, which contained about 84,000

lineal feet of logs, were sunk in sections 19 feet wide and 20 feet long. They were sheathed up to about 3 feet above low water, with sheet-piling in three layers, on the Wakefield system. The driving was accomplished by attaching an eighty-pound weight to an Ingersoll-Sergeant drill run by steam and utilizing the reciprocating motion by attaching the drill with clamps to the tops of the sheathing, following it down as it was driven, after the manner of the Nasmyth steam pile-hammer.¹ This device, which is one of the most ingenious ever devised for the purpose, was arranged by the contractor's engineer, S. H. Reynolds, and was a complete success.

The tops of the cribs were 10 feet above low water, and the bottoms rested on the hard-pan, making a total height of from 30 to 34 feet.

The cribs were filled with sand and gravel that had been dredged out, but the outside was banked up with selected clay and dredged material, which was protected by a layer of riprap up to about low water.

When the coffer-dam was first pumped out several leaks were developed, but after one week in perfecting the details the pumps were started regularly and no serious trouble was had afterward. This is only one of a series of coffer-dams which have been constructed on the several dams in this river, and owing to the care exercised good results were obtained uniformly.

The construction of a similar piece of work on the Ohio River was begun by Major R. L. Hoxie, corps of engineers, and is described in the report of 1895: "It was originally planned to inclose the site of the dam and lock within a coffer-dam, and work was commenced upon that basis. But on attempting to pump out the inclosure it was found that water came in in large quantities, not only under the dam but from springs in the bottom, and all attempts to close these by dumping clay and gravel was a failure. The area inclosed by the dam was about 600 by 200 feet or about 3 acres of river bottom. The deposit of sand and gravel overlying the rock was about 35 feet thick, the rock being 45 feet below the water-level, while the plans required an exca-

vation 20 feet deep below this water-surface. The bottom deposit had been worked over for years by sand-diggers, who threw back the large stones and coarse gravel after removing the fine sand, this work resulting in a very permeable bottom, with possible channels of comparatively large dimensions extending to unknown distances beyond the limits of the coffer-dams."

This is perhaps the most frequent source of failure of a well-constructed coffer-dam and should always be guarded against

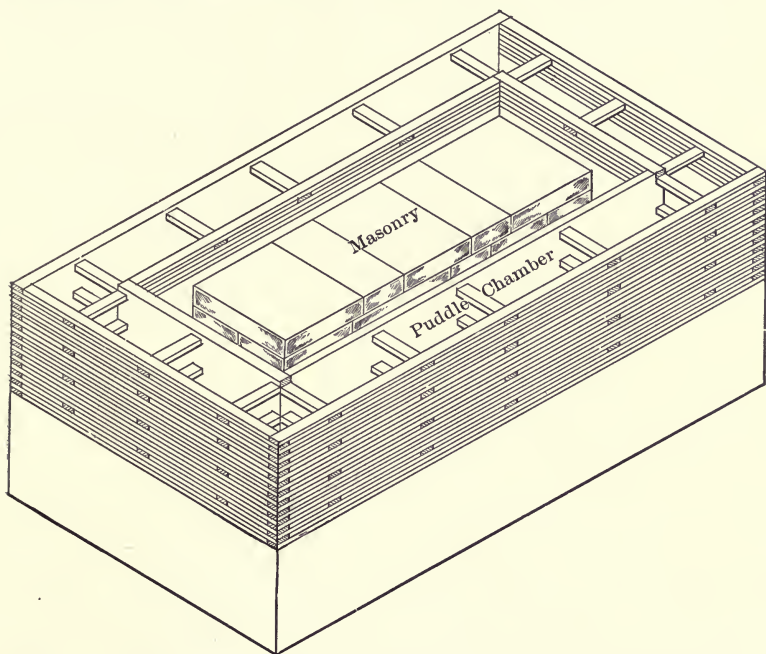


FIG. 10.—CRIB COFFER-DAM, CHICAGO, BURLINGTON AND QUINCY RAILROAD.

by removing as much of the porous material as possible, by dredging, before the construction of the coffer-dam is begun.

Cribs are very easy to construct, usually very substantial, and easy to make use of by floating to position and then sinking in place. A very simple form that has been used on the Chicago, Burlington & Quincy Railroad is described by E. J. Blake, chief engineer. Where the water is shallow they have been built in the form shown (Fig. 10), of fence boards spiked one piece on

another; with deeper water they are made of heavier timber, $2'' \times 8''$ or $2'' \times 10''$. They are built on the water and are tied across at intervals by pieces spiked through the wall, which pieces should be carefully fitted to prevent leakage. In some cases where the bottom is soft, instead of dredging, a bottom is added to the crib to prevent the filling from squeezing its way out from under the edge.

When the crib has reached bottom, being sunk by weighting it down if necessary, the chambers are filled with clay puddle and clay is banked up around the outside to prevent water running under. The crib is made large enough so that the excavation will leave an easy slope to the inner edge of the timber work. This form can be made to conform readily to the contour of the bottom by starting the layers of timber at different elevations. No leakage has been experienced except what can readily be kept under control with ordinary-sized centrifugal pumps. The cost of construction is generally a minimum, as there are usually plenty of old timbers available for use from the railroad yards.

Cribs constructed in a similar manner but with only one wall of timber have been used successfully on the Canadian Pacific Railway by P. Alex. Peterson, chief engineer.

The bracing is very efficiently attached by dovetailing it into the sides, while the form of the crib enables it to withstand the force of the current and the ice. The projections on the inside are to prevent the water from forcing its way up between the sides and the concrete filling when the dam is pumped out. These projections answered their purpose very effectually, and when the dam was pumped out it remained dry enough to lay the masonry without any additional pumping.

Illustrations are given of a crib of this character which was used on the St. Lawrence River (Fig. 11), similar ones being used for the other piers of the same bridge, and of the crib used for the Arnprior bridge. (Figs. 12, 12a.) This shows the concrete which was deposited on which to found the masonry, and which formed a water-tight bottom so that the crib could be pumped out for the laying of the stone.

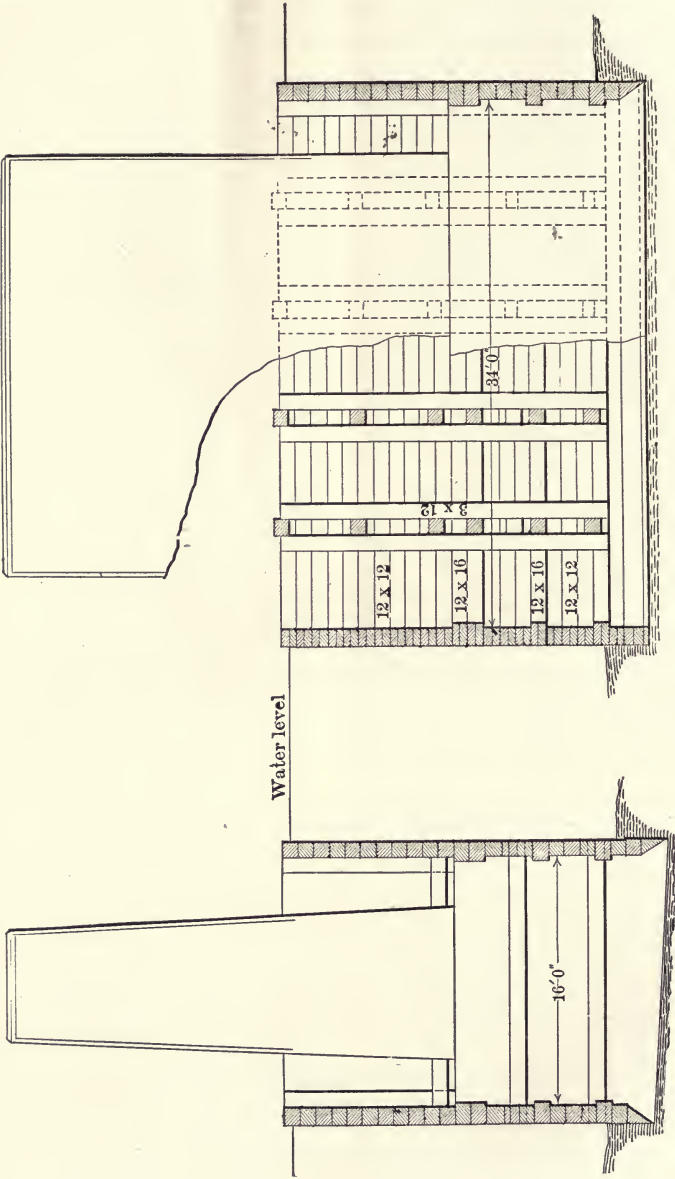


FIG. 12a.

encountered a crib is made by constructing a frame of old bridge timbers and sheathing it with plank. (Fig. 13.) This is sunk by digging out the sand, which is shoveled first into box A, then to boxes B, then to C, and then outside. The suction-pipe is shown in dotted lines, the pumping being accomplished with a centrifugal pump. This plan works very successfully on the streams in Colorado and New Mexico where the water is mostly in the sand and but little shows as surface-water.

The Arkansas River bridge of the St. Louis & San Francisco Railroad at Tulsa was built over a bottom of gravel and riprap above rock, which was quite level and about 7 feet below low water. Cribs were constructed for coffer-dams similar to the one just described and set on the bed of the stream. Clay from the bank was dumped outside and as the crib was dug out and sunk, the clay followed down and kept out the water.

When the bottom is of clay or of sand without obstructions, sheet-piles, either tongue and groove or the Wakefield, are driven around a crib.

Geo. H. Pegram, chief engineer of the Union Pacific system, has made the construction of coffer-dams conform to available material and local conditions. At the crossing of the Republican River in Kansas, where the bottom was sandy, a single thickness of 4-inch V-shaped tongue-and-groove sheet-piling, with the usual guide-piles and wales, served to form a water-tight structure.

Where a gravel bottom overlaid a hard soapstone, as on some work in Idaho, with 7 feet of water to contend with, the coffer-dam was made of Wakefield piling, formed of $1\frac{1}{2}$ -inch sized plank. The joints were tightened with cement; and sand, gravel, and straw placed outside to prevent leaking. Wakefield piling has also been used for clean rock bottom, placed in two rows about the depth of the water apart. Intermediate cribs filled with rock were used to sink them. The ends of the piling were sharpened and driven on the rock until broomed up and rendered nearly water-tight, when gravel mixed with straw was placed around outside to close any remaining leaks.

In cases where ordinary piling has been driven and a grillage

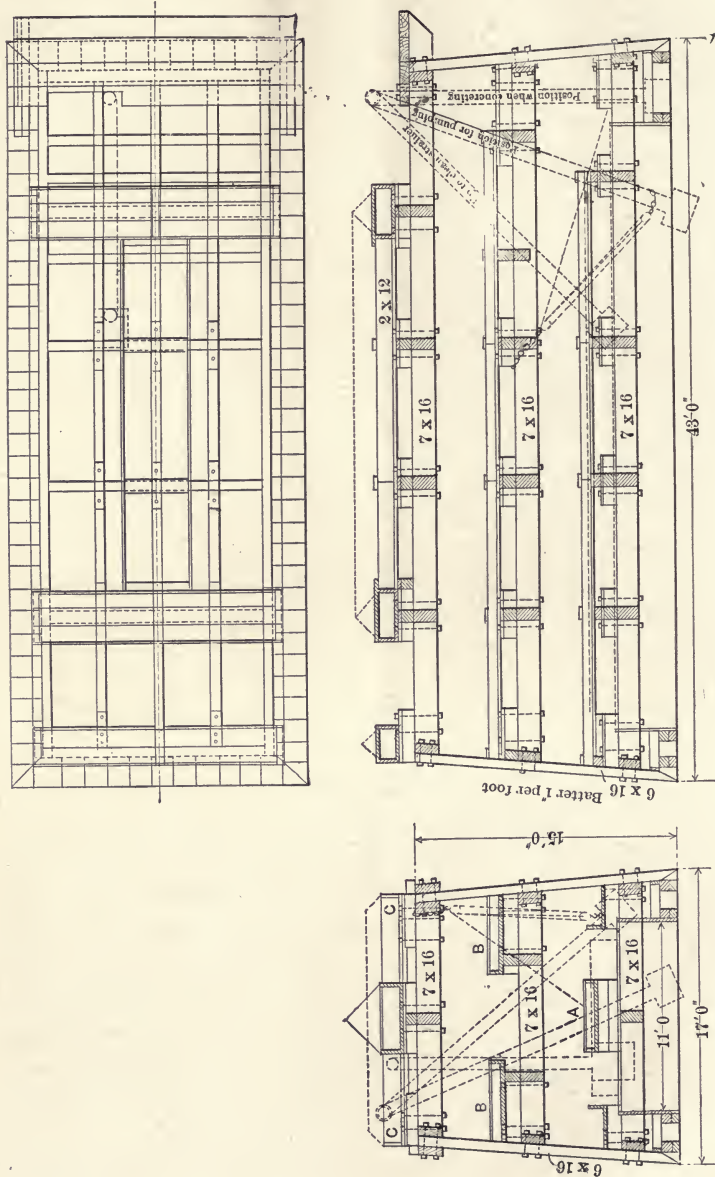


FIG. 13.—CRIB COFFER-DAM, ATCHISON, TOPEKA AND SANTA FÉ RAILWAY.

laid upon them to receive the masonry, a coffer-dam is constructed as shown (Fig. 14), in which to lay the masonry. The construction of this is fully shown in the different views given.

Another form of coffer-dam for the same purpose was constructed by Octave Chanute in laying the masonry of the pivot pier for the Fort Madison bridge over the Mississippi River, on the line of the Atchison, Topeka & Santa Fé Railroad. (Fig. 15.) This is described in the *Engine ring News* of June 2, 1888, by W. W. Curtis, resident engineer: "The grillage (for the pivot pier) is 4 feet 3 inches thick, the upper 15 inches being dressed to an accurate circle of the desired diameter. The coffer-dam was fitted against these two courses and was formed of 3"×8" pine-plank staves, dressed on the sides to a slight bevel, around which were placed seven wrought-iron hoops 4"× $\frac{3}{16}$ ", 5"× $\frac{3}{16}$ ", and 6"× $\frac{3}{16}$ ", similar to those used for water-tanks, and screwed up tight. Inside of these, circular braces of plank were fitted. As a water pressure of 19 feet was to be resisted, additional security against leakage was obtained by placing a string of candle-wicking vertically between each stave. When the caisson was submerged to about full depth it became necessary for the steamboat to assist it into final position. A 12"×12" post was bedded in the concrete in the center of the pier, with four braces running to the circular bracing of the sides. This makes a very cheap coffer-dam and was found to work very well."

An attempt to use a form similar to this was made in constructing the Walnut street bridge at Philadelphia. This is described by Geo. S. Webster, chief engineer Bureau of Surveys, in the *Engineering News* of March 15, 1894: "In founding the river piers, the Robinson coffer-dam was first tried, but was abandoned after three of them had failed by collapsing. This dam may be briefly described as follows: A circular platform about 80 feet in diameter supported upon piles at an elevation of about 4 feet above high water was first constructed. Square piles of 12"×12" yellow pine were then prepared by spiking a 3"×4" timber flat, along the middle of one side, and two others along the edges of the opposite side, forming a tongue and groove on each pile.

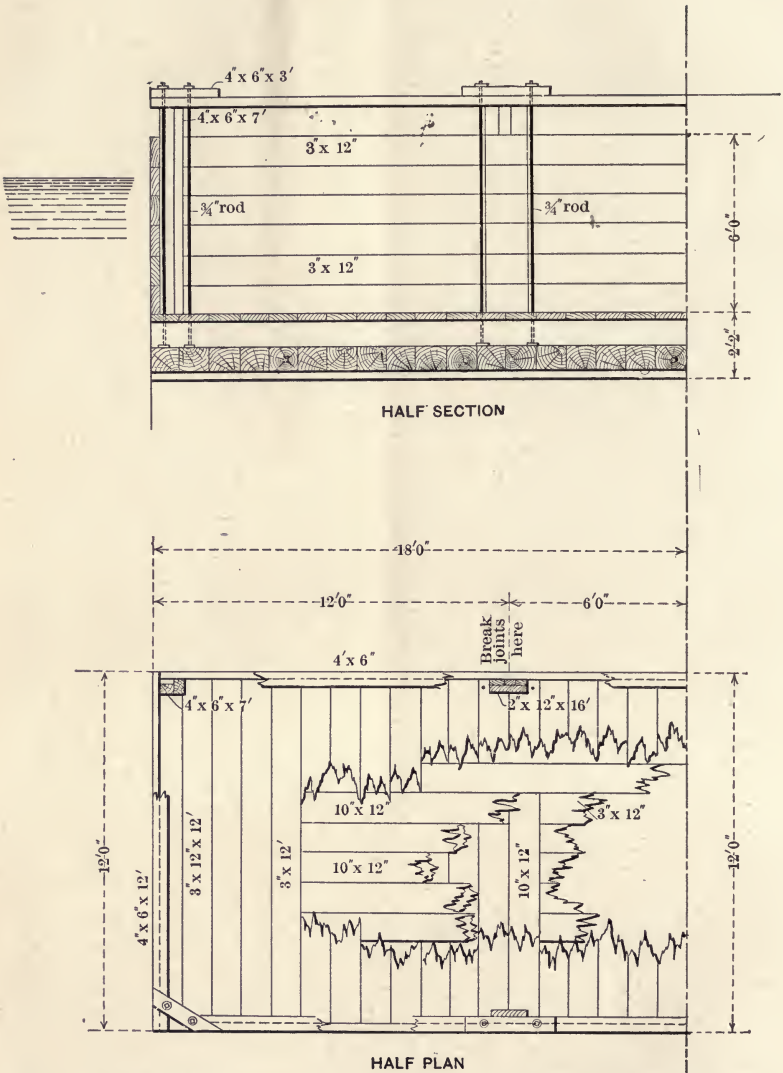


FIG. 14.—COFFER-DAM ON GRILLAGE, PAYETTE AND WEISER RIVER BRIDGES, UNION PACIFIC SYSTEM.

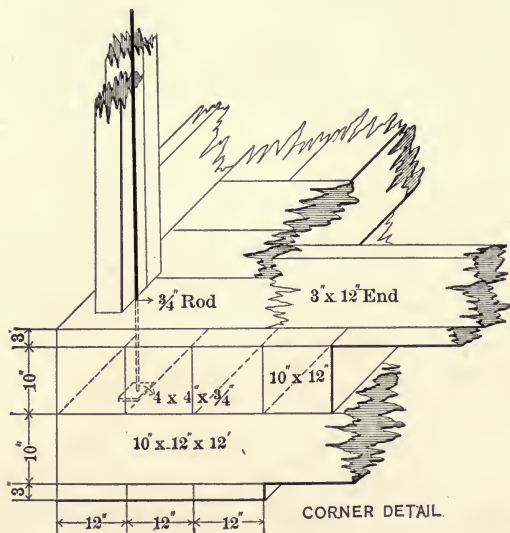
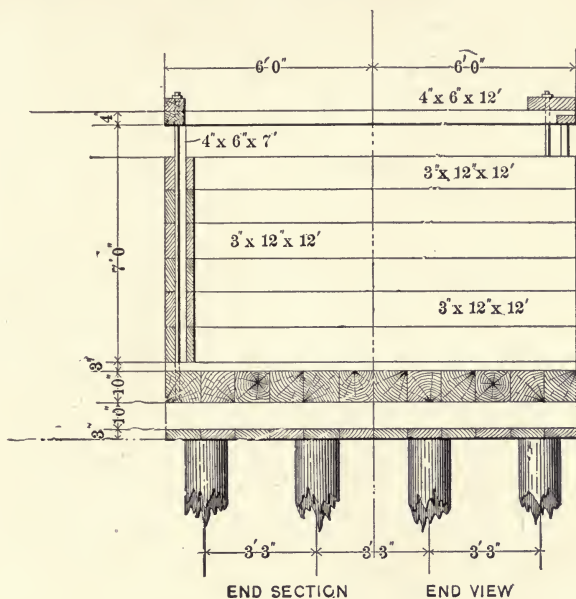


FIG. 14a.

The tops were squared off and the bottom ends pointed to a wedge shape. These piles were then driven close together against the edge of the circular platform and down to rock. Mr. Robinson's idea was that the mud overlying the rock would hold the piles in position at the bottom, and if the top ends were held by an outside hoop, the dam would be secure without internal bracing to resist collapsing-pressure. In the first trial the hoop was made of boiler-iron some 4 feet or more in width. In the second dam it was formed of a heavy steel railway rail, and in the third

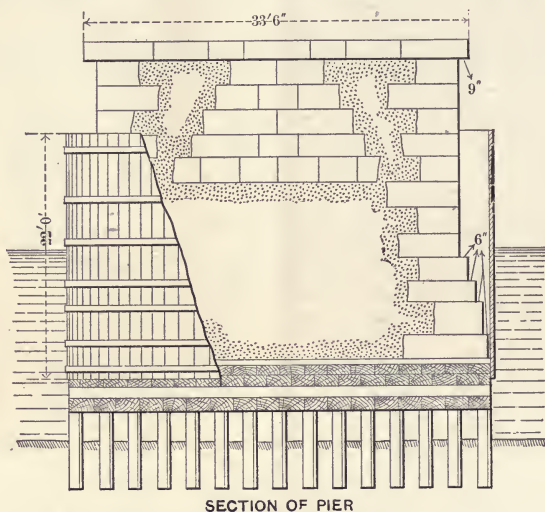


FIG. 15.—COFFER-DAM ON GRILLAGE, FORT MADISON BRIDGE, ATCHISON, TOPEKA AND SANTA FÉ RAILWAY.

dam the hoop was the same as in the second, but it also had a number of radial rods in addition. The first dam was pumped out and held for nearly an hour before collapsing, but the others collapsed before being entirely pumped out. After the third failure this form of dam was abandoned."

It would seem likely from a comparison of the two cases, one being entirely successful and the other a failure, that had the Walnut street dam been supplied with additional bands lower down and provided with some means of tightening, with several internal bracing ribs of timber, it would have proven a success. These bands and ribs could likely have been placed by a diver.

The uncertainty which always exists regarding any construction under water makes it imperative that every precaution should be taken to guard against troubles that might arise, by making the construction of no doubtful form and in no doubtful manner from its first inception.

The nature of the bottom will always indicate the method of construction which should be adopted in a given case, but it



FIG. 16.—A CRIB COFFER-DAM AFTER A FLOOD.

would be rarely that the preliminary dredging could be dispensed with. It is true that there are cases where there is a deposit overlaying a seamy rock, and the water will find its way along the seams, bubbling up in springs inside. Recourse must be had to cutting off the flow, by puddling on the outside, sometimes extending the operations a distance of a hundred feet or more away, until enough of the flow has been stopped so that the water can be kept down by a reasonable amount of pumping.

The next precaution after dredging is the building of some form of coffer-dam which shall effectually exclude any flow through the sides of the dam. This we have seen to be accom-

plished in many cases by means of a bank of clay, or a row of sheet-piling, and in some cases by a single-walled crib. But in the last two methods a supplementary bank of clay or clayey gravel on the outside is necessary to prevent leakage.

This bank may be protected from wash by covering it with clay, sand, or gravel in gunny-sacks, or by riprapping up to about low water, as was done on the Kanawha dams.

Double-walled cribs and coffer-dams, constructed with two rows of water-tight sheet-piling, require to be puddled with a carefully selected material. While clay can be used with a good degree of success, it will be found better to use a clayey gravel or to mix the clay and gravel, as was done at the Buda Pesth bridge. When a small leak starts through a pure clay puddle, it washes out the clay in considerable quantities and a dangerous leak is soon developed. With the admixture of gravel, however, a leak is stopped almost as quick as started by the heavier gravel falling into and closing the void.

It will generally be found advantageous to use a bank of clay outside of a double-walled dam, unless it might be a case where sheet-piling has been driven to rock, and even then a certain amount of material in sacks should be used to prevent wash or the cutting out of the earth around the sheeting.

Whatever excavation is taken out of the interior of the coffer-dam after it has been pumped, should be dumped at the up-stream end and corners, or to fill any holes or pockets there may be around the sides or ends.

Cutwaters should be added to all coffer-dams which are built in rivers having a swift current or a heavy flow of ice, as was the case at Buda Pesth and on the Canadian Pacific examples. They must also be used in rivers where the run of drift with each rise is of large amount. For the purpose of preventing wash around a dam, a cutwater of plank supported by a frame of timber may be constructed separate from the main structure, or a V-shaped row of sheet-piling driven up-stream. On rock, a timber crib of triangular shape, built of round logs, may be sunk up-stream and filled with broken stone. Such a crib can be

utilized in anchoring the main crib of a coffer-dam, as was done at St. Louis, and which will be described in future pages.

More fitting language cannot be found for closing words than those used in Wellington's monumental work on railway location: "The uncertainty as to the exact requirements to be fulfilled by the works when completed is a disadvantage, indeed, which cannot be escaped; but the more difficult it is to reach absolute correctness, the greater need we have of some guide which shall reduce the unavoidable guess-work to its lowest terms, and so save us from the manifold hazards which result from not only guessing at facts, but at the effect of those facts. Whatever care we use we can never attempt with success to fix the exact point where economy ends and extravagance begins; but what we can do is to establish certain narrow limits in either direction, somewhere within which lies the truth, and anywhere outside of which lies a certainty of error."

CHAPTER III.

CONSTRUCTION AND PRACTICE.—CRIBS AND CANVAS.

WHEN for some reason the necessary care has not been exercised in the construction of a coffer-dam and in puddling it, or where there were discovered conditions not known before the construction began, which rendered the work unsatisfactory or leaky, it will usually be found that the mode of repair which seems most expensive will in the end prove the cheapest and most expeditious. If the puddle proves leaky, and it be decided that the material was of too porous a nature, the best remedy is to dig out and replace it with better. Should it be found that the porous bottom had not been removed to a sufficient depth, it may be found necessary to dig out the puddle-chambers and puddle deeper, or the leaks might be stopped by banking up outside of the dam with clay or clayey gravel, or perhaps sand in sacks would do some good.

Gravel will allow the percolation of water even where the head is small, and when a pressure of from 4 feet upwards is brought upon it, the leakage becomes considerable and difficult to control, so that pure gravel is of little service in stopping leaks.

Hay, straw, oats, crushed cane-stalks, rotten stable manure, and similar materials, mixed with the banking material, are very efficacious in producing tightness, and when applied to local leaks will assist in closing them.

Where sheet-piling has been used to exclude the water and leaks still occur, they can often be closed by driving more sheeting to lap the cracks, which may have been widened out lower down as the sheet-piles were first driven. This, we have seen, pro-

duced satisfactory results at Buda Pesth, where leaks were also closed by driving square timbers into the puddle to compact it.

Clay can also be forced down through pipes directly to where the leakage occurs. The use of this at the Government Lock at Sault Ste. Marie is described in the *Engineering News* of September 26, 1896: "The only difficulty encountered in the work of excavation was due to a leak in the coffer-dam, which flooded the lock-pit and delayed the work considerably. The cause of this leak was found to be a crevice in the rock passing underneath the coffer-dam, and despite all efforts to close it, the flow of water rapidly enlarged the break until about 50 feet of the clay in the coffer-dam had been washed away. The large break was closed by driving additional sheet-piling and filling in with brush, hay, and clay in sacks. This, however, failed to entirely stop the leak through the crevice, and it was determined to fill the cavity with clay. For this purpose a 3 inch pipe was driven down through the coffer-dam until its lower end penetrated the crevice. In this pipe small cylinders of clay about one foot long were placed and forced down into the cavity by means of a plunger working in the pipe. The apparatus is shown in the illustration (Fig. 17). As will be seen, the plunger, or rammer, is an iron rod, to the top of which is fastened a block of wood sliding between the guides of an ordinary pile-driver. The hammer of the pile-driver is the weight which pushes down the rammer. This apparatus was designed by E. S. Wheeler, engineer in charge of the work, and was used not only to fill the crevice, but all along the coffer-dam for the purpose of compacting the clay filling. The apparatus proved most successful for the purpose for which it was intended."

The use of rods or bracing in double-walled coffer-dams is very often the cause of considerable leakage, the water following along them through the puddle. This may be stopped by wrapping a band of hay or straw around the rod next to the timbers, or by a wrapping of coarse cloth, or by a wood washer having a hole slightly smaller than the rod, which is forced through.

The walls of the dam must always be made tight, and this we have seen to be effected by careful framing of sides and bracing,

and it will be seen in a later example how round struts between the two walls allowed the puddle to flow around them and close up much better than if the braces were square timbers.

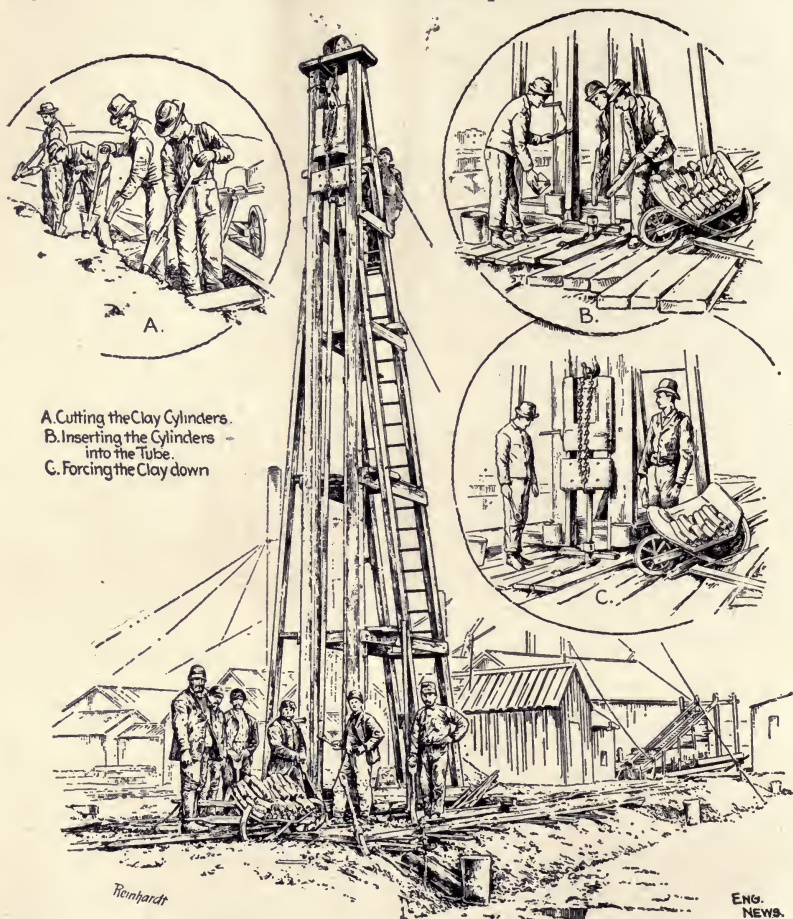


FIG. 17.—APPARATUS USED TO FORCE CLAY INTO CREVICE OF FOUNDATION ROCK AND CLOSE LEAK IN COFFER-DAM.

The use of candle-wicking between the staves proved successful at Fort Madison, and calking is very often resorted to at the first, and also to close up local leaks. The use of this and the use of a stiff grease between the layers of a crib will be referred to in another part of this article.

The use of tarpaulins to make a water-tight piece of work is described in the Trans. Am. Soc. C. E., Vol. 31, by Montgomery Meigs, engineer in charge of the government work at Keokuk, Iowa. "The upper one of three locks was twice repaired by separating it from the river by an ordinary plank and mud coffer-

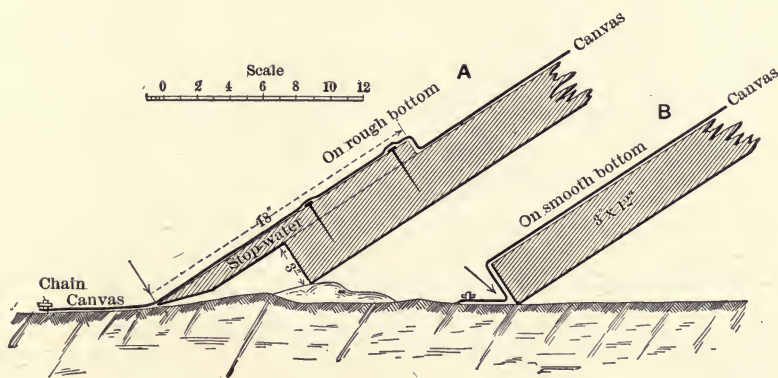
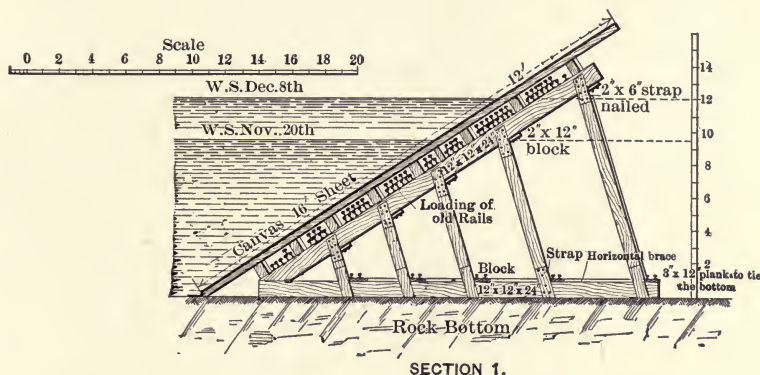


FIG. 18.—DETAILS OF CANVAS AND PLANK BULKHEAD.

dam. But as this work had to be done after the close of navigation, it was found to be very unsatisfactory on account of the freezing of the puddle, and on one occasion the partly puddled dam froze and upset. After this experience it was determined to use some other method than puddle to produce tightness. There was available for drainage a 50-H.-P. suction dredge, with 14-inch

suction, and a rotary Van Wie pump, and plenty of 12-inch discharge-pipe mounted on pontoons. It was proposed to drain the lock with this dredge, allowing the boat to settle in the mud at the bottom of the lock as the water left it, and to complete the work with a 3-inch discharge pulsometer. The lock being 350 feet long and 80 feet wide, a flat place on the bottom was selected, the dredge placed over it and the necessary length of discharge-pipe placed in position on its pontoons. The point selected for a bulkhead (Figs. 18 and 19) was just outside the lock gates, about 40 feet below the lower mitre-sill, where there was a smooth rock bottom, the ends of the dam abutting against the flaring ashlar wing-walls of the lock approach.

"The bulkhead was constructed with thirteen bents 8 feet apart, of the size timber shown, with light diagonal bracing. After being built $2\frac{1}{2}$ miles from the lock it was towed to position and sunk by weighting it with old railroad-rails, enough being used to overcome the buoyancy after the sheathing was added. A diver was employed to see that the bottom was clear of obstructions and to guide the bulkhead to a solid bearing. The sheathing was also guided to place by his assistance.

"The canvas sheet, which was designed to give tightness to the apron, was of two breadths of 10 feet and one breadth of 6 feet wide, sewed together edge to edge for convenience, and about 4 feet longer than the extreme length of the apron. Some old $\frac{1}{2}$ -inch and $\frac{3}{8}$ -inch chain was sewed to one edge continuously to act as a sinker and insure the lower edge of the canvas sheet hugging the bottom tightly. A few stones laid on it would have answered the same purpose, but not so well. The canvas was 12-ounce duck.

"The sheet was spread under water by the diver. It lapped on the bottom about 12 inches, covered the face of the apron and extended some inches up the face of the wing-walls at the end of the dam. Cleats were nailed on the angle between the apron and the wing-walls. These were of 1×4-inch strips, nailed with 2-inch wire nails about 12 inches apart. The upper edge of the canvas was also lightly cleated to the planking in a

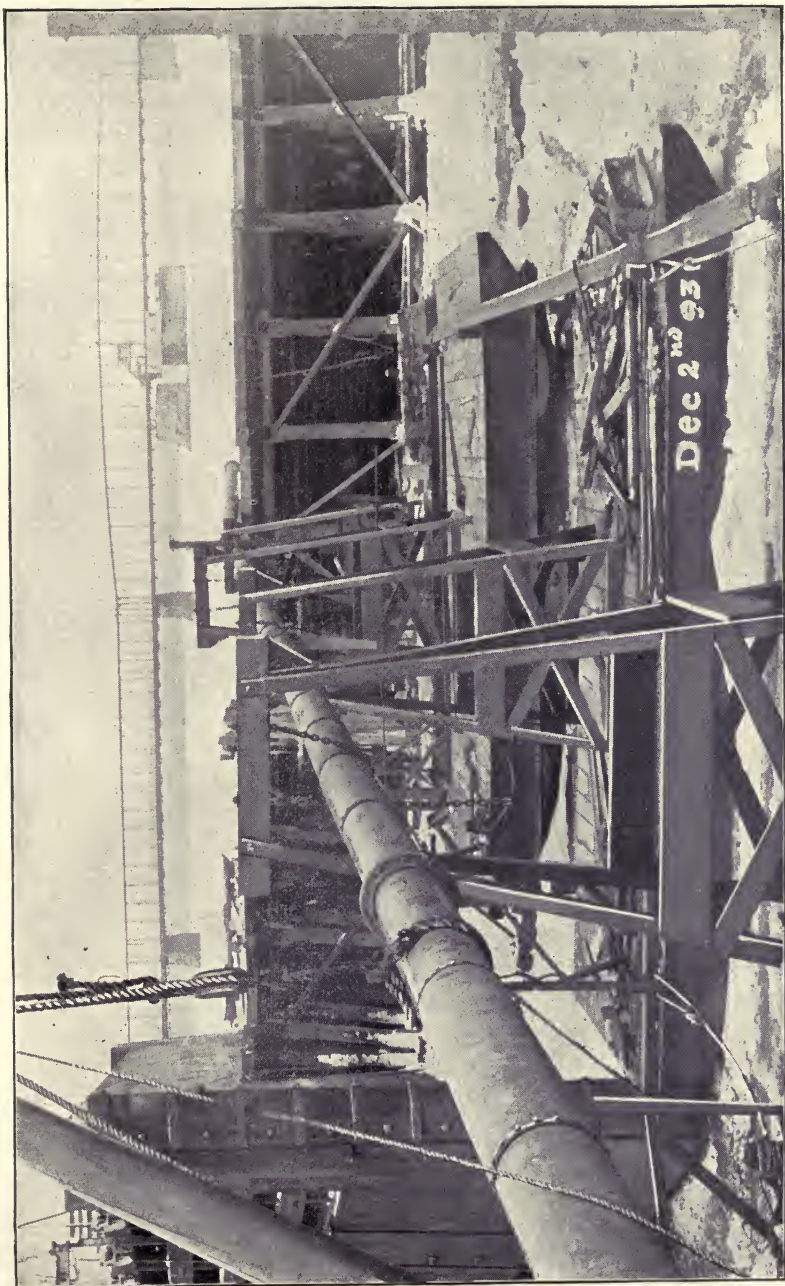


FIG. 19.—INSIDE VIEW OF BULKHEAD. LOCK PUMPED DRY.

similar manner. No other nails were driven in the canvas, which was designed to be cut up into tarpaulins eventually. Where the plank touched bottom no beveling was used, but one ragged hole was stopped with the beveled 'stop waters' which were made use of. The dam was pumped out in about 6 hours and the leakage was so small that a 3-inch discharge pulsometer kept out the water, and was then run only at intervals. Small leaks were stopped by dumping rotten stable manure in their vicinity."

It is interesting to note that the bulkhead stood a pressure of 12 feet of water. Experiments made to determine what pressure 12-ounce duck would stand, show that the clean canvas begins to leak at 2 pounds pressure, and at 5 pounds pressure the leakage becomes a marked amount. With mud on the can-

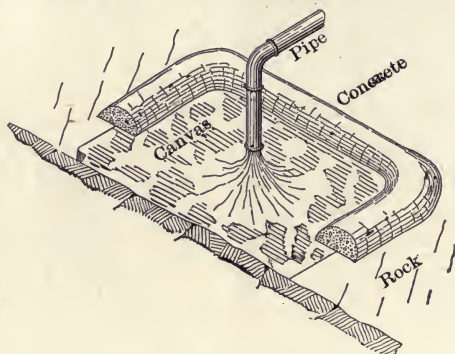


FIG. 20.—CANVAS FUNNEL FOR CLOSING LEAKS.

vas the leakage becomes noticeable at from 5 to 7 pounds, and of a considerable amount at 50 pounds pressure, these pressures being on a circle $4\frac{1}{2}$ inches in diameter. The canvas did not rupture at 800 pounds.

The suggestion is made to use an inverted funnel of canvas to stop the leakage of springs on rock bottom. (Fig. 20.) The canvas to be spread out over the bottom and weighted down with concrete, and the top wired to a pipe into which the water may rise until the pressure-head is overcome or the pipe can be plugged. Arrangements of this nature, but without the canvas funnel, have been frequently used. An iron pipe set on end is

fitted over the leak, and after concreting around to make it water-tight, the water rises inside until the pressure is balanced. A water-tight wooden box may also be used for the same purpose.

The founding of a new inlet tower in the Mississippi at the St. Louis water-works was accomplished by using a coffer-dam and it was the intention to form a junction with the bottom by using a canvas curtain. When the coffer-dam was floated into position and the divers were sent down to spread the canvas and weight it down with stones, it was found to be damaged so as to be useless. This was supposed to be due to the action of the swift current, but was most probably due to some accident such as fouling on a snag or against a barge.

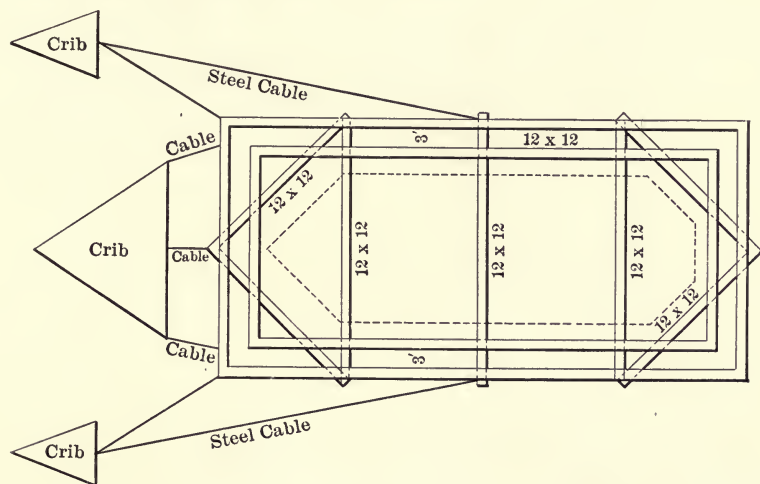


FIG. 21.—CRIBS FOR ANCHORING ST. LOUIS COFFER-DAM.

The anchoring of the crib for this dam is related in the *Engineering News* of July 4, 1891. The dam was to be located near the head of a stone dike about 20 feet in height and on solid rock bottom which was uneven and worn into grooves by the action of the current, which had a velocity of between 6 and 8 miles per hour. The bottom was leveled off by blasting, to receive the crib, which was to be sunk in from 15 to 18 feet of water.

The three triangular cribs shown (Fig. 21) were sunk and

filled with stone and were used to hold the dam in place while building and while being sunk. Steel cables $1\frac{1}{2}$ inches in diameter were used as anchors.

The large crib also served as a protection from the current and drift.

The size of the crib was 38×74 feet outside and the height 22 feet. The 12×12 -inch yellow pine timbers were drift-bolted together with from 1 to 2 feet spacing of bolts, and all the joints between the timbers were calked. The bracing consisted of 12-inch square timbers, of which there were three rows, the braces in each row being 4 feet apart vertically. These were cut out as the masonry was built up and bracing against the stone work substituted.

There were four sets of diagonal bracing as shown. The space between the walls, which was 3 feet, was partly filled with concrete in sacks, and puddle placed on top. Sacks of clay were banked up around the outside, and then the dam was pumped dry with a 10-inch pump. Inside was found 8 feet of mud and 60 sacks of concrete which had been washed there by the swift current.

The amount of timber used was 125,000 feet, B. M., and about 12,000 feet of $\frac{7}{8}$ -inch round iron for drift-bolts. The puddle-chamber required 1,000 sacks of concrete and 100 barge loads of clay, while 10,000 sacks were used for banking up clay on the outside. This work was constructed under the direction of C. V. Mersereau, Division Engineer, under S. B. Russell, Principal Assistant Engineer.

The Queen's Bridge at Melbourne, Australia, is a plate-girder structure with four piers of 8 cylinders each. The bottom was a reef of bluestone which had been shattered by blasting and which was silted over with about 3 feet of very soft silt.

The use of ordinary puddle coffer-dams was thought to be too expensive as the bridge was 100 feet in width, and it was proposed to use a single wall of timber protected by tarpaulins. The account of this work is taken from the *Engineering News* of April 4, 1895, which is an abstract of a paper by W. R. Renwick, engineer in charge.

To insure as light a construction as possible experiments were made on the strength of Oregon pine, and it was found that tests of water-soaked timber showed a loss of strength of as much as 33 per cent., when compared with tests of seasoned timber. The break, too, of the water-soaked pieces was very short. This strength being the one adopted, a very low factor of safety was used. A separate dam was constructed around each tube, but with one side to open as a door to allow its removal and use for another place. The frame was made from 12×12 Oregon pine, with the sticks placed closer together near the bottom to resist the greater water pressure, and 12×12 pieces were run up the corners, the frames being notched in. These also served as spacers for the side timbers and as door frames. The sheeting on the outside was of 4×12 rough timber, and outside of this at the top and bottom were wale-pieces, 6×12 , bolted through the frames with 1-inch bolts to hold the sheeting in place.

The tarpaulin was passed completely around the dam, being tacked to the waling-pieces, and so arranged as to allow the door to open.

When the dam had been placed around a tube the sheeting was driven down to rock, through puddle which had been dumped on the bottom, and the pumping was readily done with pulsometer pumps. The only serious leaking was where the 1-inch bolts passed through the joints between the sheeting, but these were plugged with soft wood plugs, and in other work the bolts were flattened to three-eighths of an inch where they passed between the plank. The dams were removed by first drawing the sheeting up to its original position, when the door was opened and the crib taken to another tube. The depth of water was about 15 feet, but while this was successful in this instance, the method should not be copied unless the conditions are favorable, nor unless the cribs are made practically water-tight in themselves.

This was the case in the above work, as one of the tarpaulins was accidentally torn off and the dam still excluded the water, so that the tarpaulin was only a wise precaution. Why the cylinders were not made water-tight and used as their own

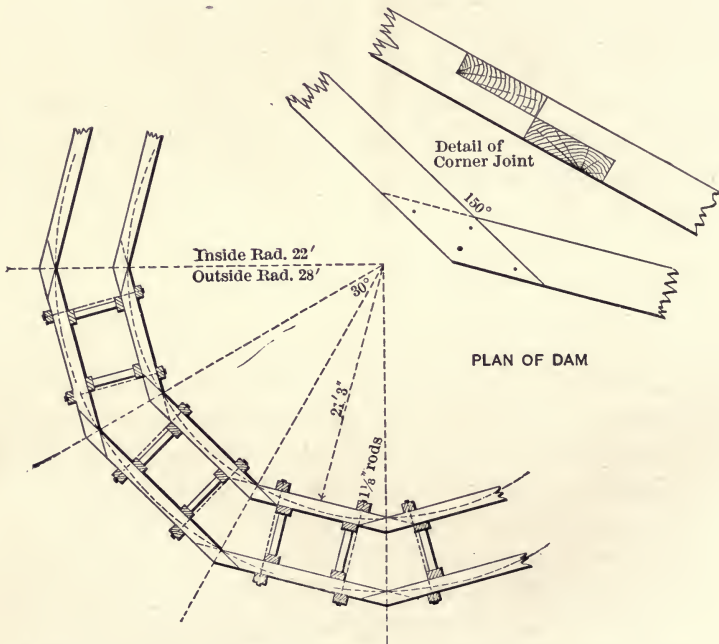
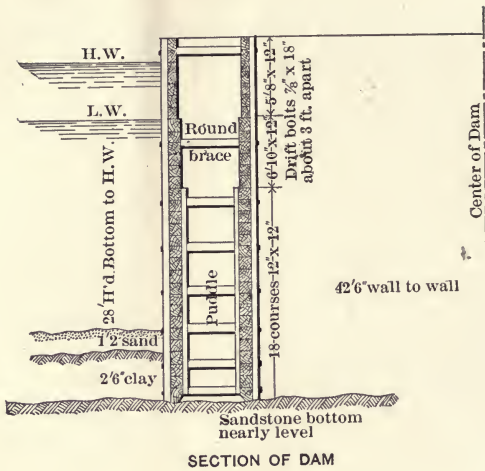


FIG. 22.—DETAILS OF COFFER-DAM USED ON ARTHUR KILL DRAWBRIDGE.

coffer-dam is not stated, but this possibly could have been done.

The use of tarpaulin in closing accidental leaks could doubtless be made use of frequently, but as the sole dependence for producing tightness it should be used with extreme care, in a gentle current and well protected from damage.

The pivot pier of the Harlem Ship Canal bridge was founded in a polygonal coffer-dam, from the plans of William H. Burr, consulting engineer. The work is described in the *Engineering Record* of July 24, 1897: "The rock bottom secured by the canal excavation being an acceptable surface for the masonry of the pivot pier, it was constructed in a polygonal double-walled coffer-dam with thirteen sides 25 feet high and 60 feet in extreme diameter. The great dimensions of the coffer-dam would have made it difficult to build and launch it on shore. Consequently it was built partly on a detachable raft. As shown in the illustration (Fig. 23) the inside wall was built up of timbers lapped and halved at the angles; the outer wall timbers were carefully butt-jointed and secured by cross-struts and 1-inch bolts to the inside walls. The rough-sawed horizontal surfaces of the inner wall were bedded in stiff grease and the joints calked, which notably resisted the penetration of the water. Each course of timber was secured to the one below it by $\frac{3}{4}$ -inch drift-bolts spaced about 4 feet apart. When the bottom was thoroughly cleaned the concrete was dumped in place by a special steel bucket. Concreting was carried on night and day and was completed before puddling was begun. Considerable difficulty was occasioned by the irregularities of the bottom which the coffer-dam could not be made to fit closely. Divers were sent down and filled in bags of sand, as at *S*, and riprap *R* was piled up outside to protect it. Then the space between the walls was filled with puddle."

Another polygonal dam was constructed for the draw pier of the Arthur Kill bridge, by Alfred P. Boller, consulting engineer. The following account is taken from Vol. 27 of the Transactions Am. Soc. C. E.: "It was necessary to use as little space as possible for the dam, and to construct it without interior bracing, so

that a double-walled twelve-sided polygon (Fig. 22) with walls 4 feet apart in the clear was used. The rock bottom was overlaid with 2 feet of clay and the clay with 18 inches of sand and mud, the depth of water over the rock being 28 feet at high tide. The square hemlock timbers used in the walls were halved

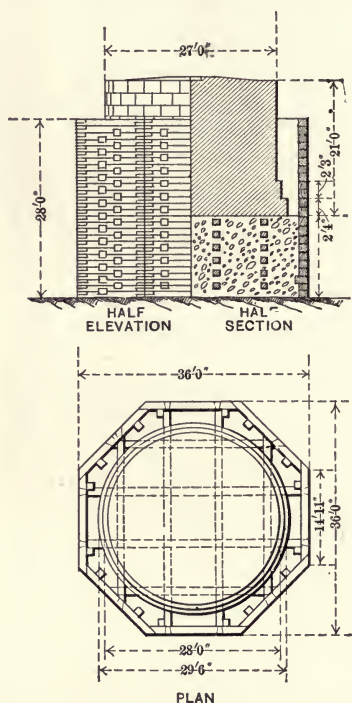
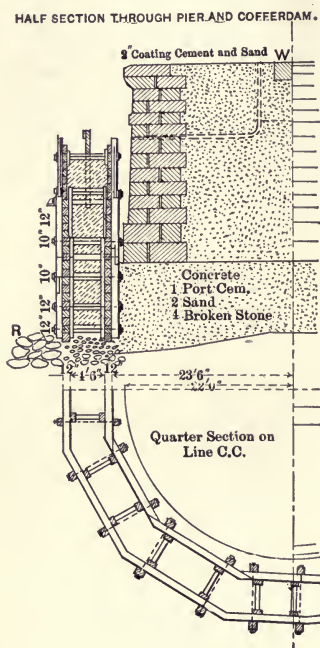


FIG. 23.—POLYGONAL COFFER-DAM, HARLEM SHIP-CANAL DRAWBRIDGE.

FIG. 24.—COFFER-DAM FOR PIVOT PIER OF THE COTEAU BRIDGE.

together and the walls braced together by bolts and round timbers for struts, the round timbers allowing the puddle to run around them and pack well as thrown in. Clamp timbers 4×6, in two lengths, were held in place by the bolts and the struts were braced against 6-inch plank. The dam was built to one-third its height on shore, then towed to position and built up until grounded. Between the timbers and the joints candle-wicking was placed, and the courses drift-bolted together every 3 feet and spiked at the joints. The rock was dredged bare before placing the crib,

which was filled with a hard, gravelly clay between the walls after being sunk in place. A rich Portland concrete was dumped inside, from triangular buckets, to seal the bottom and then the dam was pumped out with a 6-inch pump and kept dry by pumping at intervals. In one place the concrete was not thick enough and a spring came up through a fissure in the rock. This was boxed in and led to the sump. The material used was 140,000 feet of timber, 15,000 pounds of iron, and 600 yards of puddle.”

A piece of work similar to the Canadian Pacific example was an octagonal single-walled dam used in the construction of the Coteau bridge on the Canada Atlantic Railway. This is illustrated in the *Engineering News* of May 30, 1891 (Fig. 24), and was braced thoroughly with cross-timbers built into the sides. The bottom being of rock it was partly filled with concrete to make it water-tight.

The different forms of sheet-piling will next be taken up, together with pile-driving machinery and the methods of driving both sheet- and guide-piles. After this will be described the use of sheet-piles for forming water-tight coffer-dams, by reference to actual constructions of that character.

CHAPTER IV.

PILE-DRIVING AND SHEET-PILES.*

IN no department of engineering have ancient methods been more rigidly adhered to than in that of pile-driving. The form of the pile-driver derrick has remained so characteristic that a person but slightly familiar with the subject would have little difficulty in recognizing the pile-driver in the picture of Cæsar's Bridge (Fig. 3) in the first article. The bridge of the Emperor Trajan over the river Danube is an instance of the early use of piles. This bridge was constructed in the first century, and when the piles under water were examined in the eighteenth century they were found in some cases to have become petrified to a depth of three-fourths of an inch from the surface, beyond which the timber was in its original state. Before derricks were used it is probable that piles were driven by a large maul of hard wood, which is termed by Cresy a "three-handed beetle." The block of hard wood was hooped with iron and had two handles radiating from its center, to be worked by two men, while a third man assisted in lifting it by means of a short handle opposite.

Wooden mauls are still used where sheet-piling is to be driven into a soft bottom, and heavy iron mauls or sledges are also used;

* The subject of pile-driving has been restricted to the ordinary methods and operations; such unusual processes as gunpowder pile-driving and the like have not been referred to.

Pile-driving, with the assistance of the water-jet, has been described on page 80, in the account of the Sandy Lake coffer-dam. The ordinary operations of pile-driving, as practiced on that work, are also described in some detail.

but as has been frequently stated such a soft bottom should be dredged and some more elaborate apparatus used to drive the piles into a harder substratum.

The most primitive form of the pile-driving derrick is similar to the one used in 1751 by the celebrated French engineer, Perronet, at the bridge of Orleans (Fig. 25). This was arranged with a number of small ropes splayed out from the end of the lead line, so that a number of men could pull down at one time, the drop of the hammer, of course, being limited by the reach of the men's arms. The windlass shown was for the purpose of raising the pile into place between the leads.

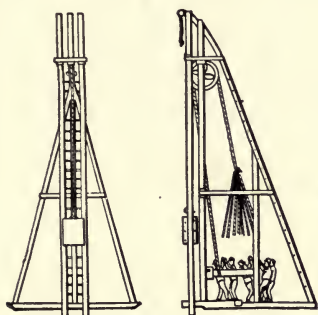


FIG. 25.—PERRONET'S PILE-DRIVER.

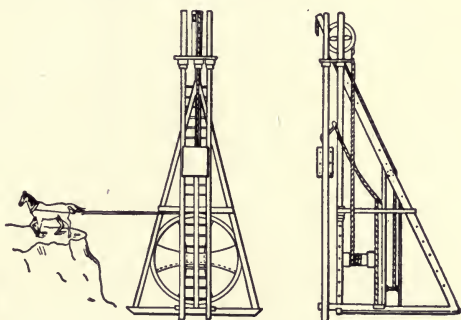


FIG. 26.—PERRONET'S BULL-WHEEL PILE-DRIVER.

The same engineer improved upon this derrick by adding a large bull-wheel to the windlass, on which was wound a rope to be pulled by a horse from the side, as shown in Fig. 26, thus winding up the lead line on the windlass. This same apparatus is in use down to the present time, except that one seen recently had the windlass at right angles to the one illustrated.

The ram or hammer used in olden times was of oak, bound with iron, and weighed for the work at Orleans 1,200 pounds for the main piles which were 9 to 12 inches in diameter and which were driven 3 to 4 feet apart, center to center, to a depth of 6 feet into the bed of the river; the ram for the sheet-piles only weighed half as much, the sheet-piles being about 12 inches wide by 4 inches thick.

At the bridge of Saumur, which was built about the year 1756, De Cessart employed a driver with a bull-wheel, in the periphery of which were set pins, to form handles for the men to pull upon and rotate the wheel. Eight men, by making three turns of the wheel, raised the ram weighing 1,500 pounds 6 feet, when it was unhooked and allowed to drop. The piles cost from two to five dollars each in place.

A very simple form of pile-driver is shown in Fig. 27 and was described in the *Engineering News* of March 16, 1893, by

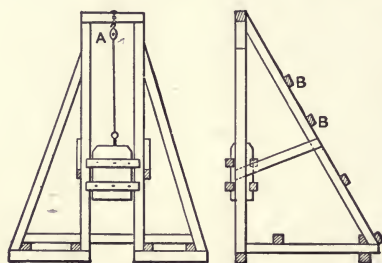


FIG. 27.—SHEET-PILE DRIVER.

Julian A. Hall. The hammer is hewed out of a section of a hardwood log, and has pieces bolted on the sides to hold it in the leads, which should give plenty of clearance. The derrick was constructed of very light timber, the verticals being 4-inch sawed stuff and the bottom timbers 6×6 inches. The rope passes over the sheave *A* and down over the tops of the steps *BB*, on which the men stand to pull the line and thus operate the hammer. This was a very inexpensive apparatus and was found to work well. Where there is already in use a heavier hammer of cast iron it can be used by striking light blows. The construction of the ordinary pile-driver derrick is a simple piece of framing, when good straight timber is easily obtained, the essential features being to keep the leads free from any obstruction for the hammer and to have efficient bracing.

For bracing a derrick under 25 feet a straight-back brace or ladder having two horizontals running to the leads, and two side-braces will be sufficient. But for a higher one, either addi-

tional long braces should be used or diagonals introduced between the leads and the ladder. The use of long braces is shown in Fig. 28, which is the design of pile-driver such as is used about

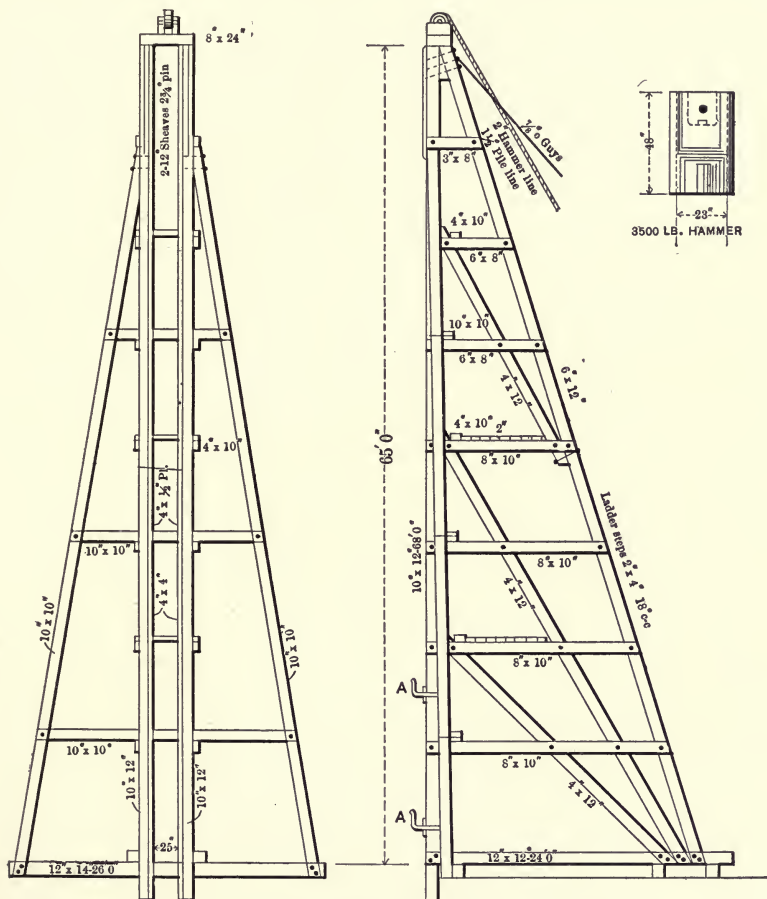


FIG. 28.—PILE-DRIVER DERRICK FOR USE ON A SCOW.

harbors or rivers on heavy work. It would be mounted on a scow or flatboat 60 feet in length, 25 feet in width and of about 6 feet in depth. The design of smaller derricks can be approximated from this one, the bracing being used in proportion.

It will be noticed that the guides for the hammer are 4"×4"

lined with a steel plate. Two lines are provided, one being for the operation of the hammer and the other for pulling piles into place. Especial attention is called to the hooks at *A*, as these are seldom shown in the plan of a derrick and they are of constant use for clamping and guiding piles. A timber laid across is wedged tight against the pile to draw it to line, and can be used to correct a stick which is beginning to slant badly. Similar clamps of course are used on the opposite side of the leads.

Where a pile begins to sliver or split in driving, if the sliver is spiked down and the clamps used to hold it in place, the trouble can usually be corrected before the pile is badly damaged.

The use of diagonal bracing between the leads and ladder is shown in the Lidgerwood derrick (Fig. 29) in which a diagonal is introduced between each pair of horizontals. This form of bracing is very satisfactory and equally as good as the other method. The diagonals on a very large driver may be extended over two panels and planks spiked down to the horizontals to form a platform for the workmen. In smaller derricks the diagonal bracing is most always omitted, dependence being placed in the stiffness of the leads and the bracing from the ladder and horizontals, as was done in the derrick shown in Fig. 4.

The power for driving with a small hammer weighing from 500 to 1,500 pounds, may be furnished by laborers pulling, but this is a slow operation and horse-power is nearly always used where steam is not available. The power is furnished from a drum with a long lever, to which the horse is hitched and winds up the hammer by walking in a circle about the drum, the frame of which is firmly fastened in place. This is called a "horse-power" apparatus and works slowly, but is a cheap and satisfactory way where a very few piles are to be driven. To the hammer-line are attached the tongs or nippers, which engage the pin in the top of the hammer (Fig. 30), and when the hammer has reached the proper height it is dropped by pulling a tripping-rope and releasing the tongs, or if the hammer is hoisted to the top of the leads, the top arms of the tongs are pushed together by the wedges on the leads and the hammer released automatically. This is a slow

method on account of waiting until the tongs run down again and engage the hammer. The horse-power, of course, has a ratchet, so that the rope runs down free and usually the blows are hurried by overhauling the line. With the addition of a hoisting-engine all this is changed and pile-driving becomes one of the most stirring operations of the contractor. When the hammer is hoisted up, the friction lever is released and the hammer descends carrying the rope with it, as the tongs are done away with and the line attached directly to the hammer. A good engine man will catch

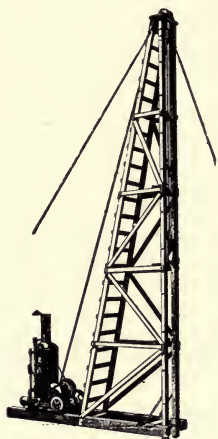


FIG. 29.—LIDGETWOOD PILE-DRIVING DERRICK.

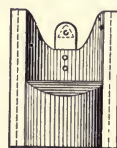


FIG. 30.—HAMMER WITH NIPPERS.

the hammer on the rebound and materially lessen the time between the blows and likewise the cost of driving.

With a heavy hammer shorter drops are made, thus causing much less damage to the pile, which would split badly under the high drop from the use of tongs. For the smaller-sized hammers—from 1,000 to 1,500 pounds—an engine of 10 horse-power is mostly used, as it is usually thought best to have a surplus of power in case of need; while for a 3,000-pound hammer a 20-horse-power engine would likely prove the best and most economical, but not infrequently a 25-horse-power hoist is employed.

The cost of an outfit will vary greatly and the only satisfactory

way is to get prices from responsible firms, but for preliminary estimates the cost of a 10-horse-power hoist with single cylinder and single drum may be taken at about \$900, and for a 20-horse-power at \$1,270. Preliminary prices for other sizes of single-cylinder, single-drum hoists, may be obtained from the formula:

$$\text{Cost} = \sqrt{81,000 \times \text{horse-power}}.$$

The double-cylinder engines will cost about 10 per cent. more and double drums about 10 per cent. additional to this.

Pile-driver derricks will vary much in cost owing to the location, on account of the cost of timber, but a minimum cost for a first-class derrick will be \$6 per vertical foot and a maximum of \$8. Being such a simple structure the easiest and safest way will be to make an estimate for each case.

In the selection of an engine it is well to remember that with a double drum a second pile may be hoisted into place, while the first one is being driven, as all derricks are, or should be, provided with two sheave wheels at the top for this purpose. While a single-drum engine has a spool for this purpose, it cannot be used very satisfactorily.

A pile-driver on a scow is shown in Fig. 31, such as was used in driving piles on the New York State canals. Another pile is just being hoisted into position. The hoisting-engine has no protection, but a shed or house is mostly provided as a protection from the weather.

While little change has ever been effected in the design of pile-driving derricks, the adoption of steam-hoists was a great improvement, as was also the invention of the steam pile-hammer by James Nasmyth. The principle is the same as that of steam forging-hammers, and was applied by Nasmyth to pile-driving in 1845, the hammers of this class bearing his name to-day. His idea was that the drop-hammer was calculated more for destruction than for useful effect and he termed it the "artillery or cannon-ball principle." Besides this the action of the drop-hammer even with the use of the "monkey" engine was somewhat slow.

Samuel Smiles says that "in Nasmyth's new and beautiful machine he applied the elastic force of steam in raising the ram or driving-block, on which, the driving-block being disengaged, its whole weight of three tons descended on the head of the pile, and the process being repeated eighty times in a minute the pile was sent home with a rapidity that was quite marvelous as compared with the old method. In forming coffer-dams for piers and

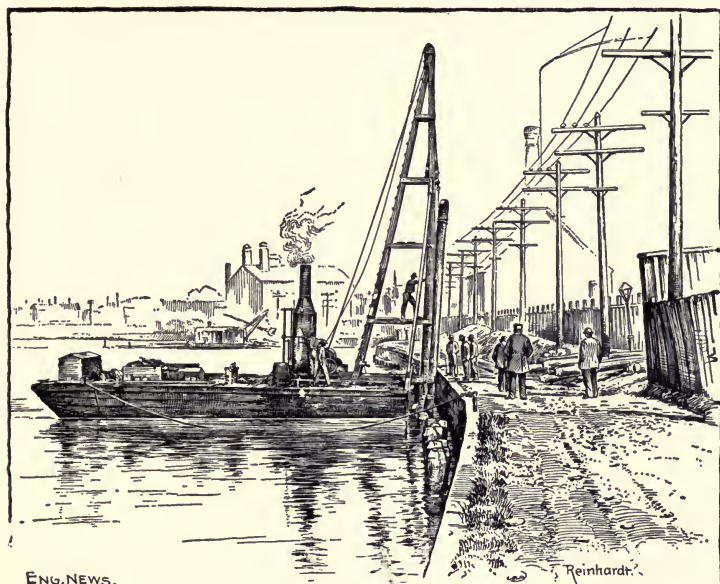


FIG. 31.—PILE-DRIVING SCOW, NEW YORK STATE CANALS.

abutments of bridges, quays, and harbors, and in piling the foundations of all kinds of masonry the steam pile-driver was found of invaluable use by the engineer. At the first experiment made with the machine Mr. Nasmyth drove a 14-inch pile 15 feet into hard ground at the rate of sixty-five blows per minute. The saving of time effected by this machine was very remarkable, the ratio being as 1 to 1,800; that is, a pile could be driven in four minutes that had before required a day. One of the peculiar features of the invention was that of employing the pile itself as the support of the steam-hammer part of the apparatus while it

was being driven, so that the pile had the percussive force of the dead weight of the hammer as well as the lively blows to induce it to sink into the ground. One of the most ingenious contrivances of the pile-driver was the use of steam as a buffer in the upper part of the cylinder, which had the effect of a recoil spring and greatly enhanced the effect of the downward blow."

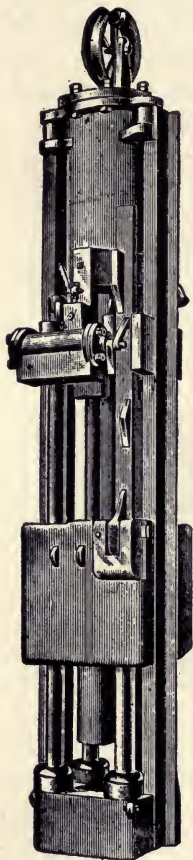


FIG. 32.—WARRINGTON-NASMYTH
STEAM PILE-HAMMER.

Many modifications of this hammer have been manufactured, and one much used at present is the Warrington-Nasmyth hammer, made by the Vulcan Iron Works. This hammer (Fig. 32) is made in three sizes, the weight of the striking parts being 550 pounds for sheet-pile work, 3,000 pounds for medium pile work, and 4,800 pounds for use on heavy work. This machine is provided with a positive valve-gear, a short steam passage to avoid the waste of steam, a wide exhaust opening to prevent back pressure as the hammer drops, a piston-head forged on the rod, and channel bars on the side to allow the pile to be driven lower than the leads of the derrick. The hammer is attached to the hoist rope, but this is left slack when the hammer is resting on the head of the pile, steam is turned on and the hammer pounds automatically at the rate of sixty to seventy blows per minute until the pile is driven. The bottom casting which rests on the pile is a bonnet which encases the top and prevents brooming or splitting.

The hammer should have plenty of play in the leads, and the steam-pipe should extend half way up the derrick to save length of hose. This hammer has a record of as high as seventy-five to one hundred piles per day, and one account gives the record of 3,000 lineal feet of piling per day at a cost of \$50, the number

of men employed being sixteen and the coal consumption one ton. This hammer is shown in Fig. 33 in use driving piles for bridge work on the Fair Haven Bridge.

Another form of the Nasmyth hammer is the Cram (Fig. 34) which is very simple in construction. The driving-head is hol-

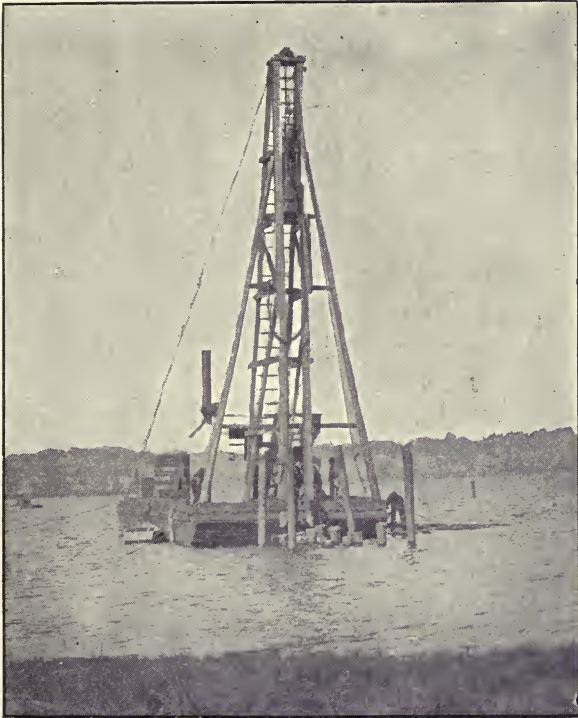


FIG. 33.—WARRINGTON-NASMYTH HAMMER, FAIR HAVEN BRIDGE.

low and the steam enters through a hollow piston-rod, causing the head or cylinder to rise on the rod. Four sizes are made, with hammers of 430 pounds, 2,000 pounds, 3,000 pounds, and 5,500 pounds. The small hammer which is listed at \$300 is used for sheet-pile work by inserting a "follower" of oak which fits the base or pile cap, and which has a slit in the lower end to fit the sheet-pile. The number of blows per minute is the same as other steam pile-hammers and an average of eighty-three piles

per day of ten hours is reported, where they were driven 17 feet into sand and oyster-shells in the Passaic River, the largest day's



FIG. 34.—CRAM-NASMYTH STEAM PILE-HAMMER.

work being 121 piles, or nearly double the best work with an ordinary hammer.

Mention has been made of the use of a rock drill as a Nasmyth hammer, on the Great Kanawha River coffer-dams; and where any amount of driving is to be done it will certainly be wise to use a hammer of the Nasmyth type.

The guide-piles of a coffer-dam should always be driven with

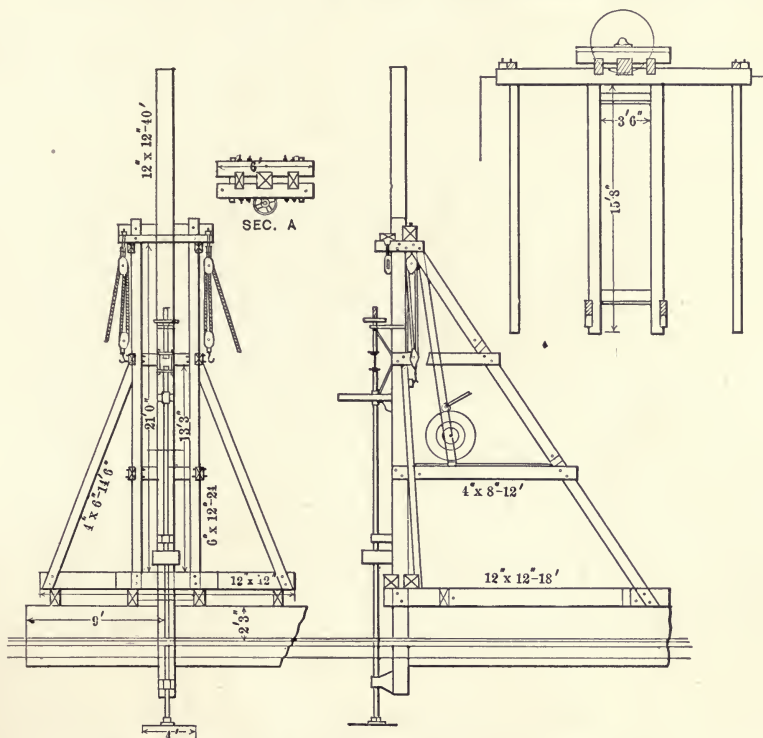


FIG. 35.—MACHINE FOR SAWING OFF PILES UNDER WATER.

the idea of using them as a support for pumps, engines, derricks, and the like, although it will often be found cheaper to rig up on flatboats when there is danger from floods. In determining what load a pile will carry from this source, or when driven as a foundation pile to support the masonry, Wellington's formula is at once the most accurate and the easiest to remember and use. For a drop-hammer, multiply twice the weight of the hammer in pounds by the drop in feet and divide by the last sinking in inches

plus one, and the result is the load in pounds the pile will carry, with a factor of six for safety. This is easily remembered as $2wh$ over $s+1$, and is always ready for use. For the steam-hammer the form is $2wh$ over $s+0.1$, the "wh" representing the dynamic effect of the hammer.

Where piles have been firmly driven and they are to be removed when the work is done they can be cut off under the water by a machine similar to Fig. 35, which can be operated from a barge. The description in the *Engineering News* gives but little information in addition to the drawing. The shaft works in cast-iron sleeves attached to a timber, which slides in the leads, this being operated by the winch shown in side elevation. The final adjustment is made by the hand-wheel on the 3-foot adjusting screw. Where the piles are not so solidly driven they can be pulled out with a lever, an old form of which is given by Cresy (Fig. 36). In place of the pin and links, a chain closely wrapped around the top of the pile is usually made use of.

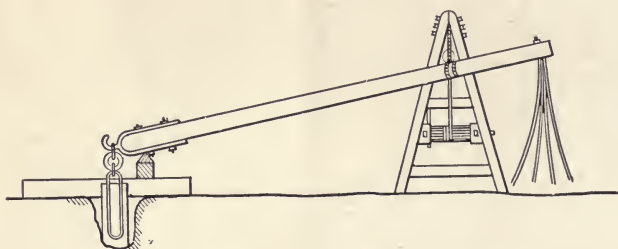


FIG. 36.—PILE-PULLING LEVER. AFTER CRESY.

The apparatus used on the New York State canal work (Fig. 37) consisted of a strong frame mounted on a scow, from which was suspended a heavy set of falls to attach to the chain wrapped around the head of the pile. The pulling was done by an engine placed on the scow.

The construction of coffer-dams with sheet-piling has led to the use of a number of forms of sheet-piles, some of which are driven only as a protection to the puddle, while others are nearly or quite water-tight in themselves. The principal forms are

shown in Fig. 38, the simplest form being plank of some considerable thickness (*a*) for which Stevenson specified $4\frac{1}{2}$ inches by not exceeding 9 inches in width for the Hutcheson Bridge. The points are sharpened as at (*i*) so they will draw together in driving, and as at (*j*) to cause them to drive straight and easy. The same principle is embodied in the patent metal point shown at (*k*), which is used to protect the point when driving through coarse gravel.

The piles at Buda Pesth were increased to 15 inches square



FIG. 37.—PILE-PULLING SCOW, NEW YORK STATE CANALS.

in order to resist the pressure brought upon the sides of the dam by the puddle, the water, and also by the ice. Flat plank are also used by driving two or more rows at as (*b*), the second and third rows being used to close the cracks in the main row of piles and retain the puddle. An example of this will be given in the next article, where it was used on the Michigan Central Railway. The extra rows may be of thinner plank if they can be driven.

Mention has already been made, incidentally, of the use of V-shaped tongue-and-groove piling (*c*), on the Union Pacific Railway. This may be made on a beveled saw-table, the saw

cutting half through the plank from opposite sides at each cut. This will produce a reasonably tight wall, if care is used in driving and if the points are sharpened to draw them together and make tight joints.

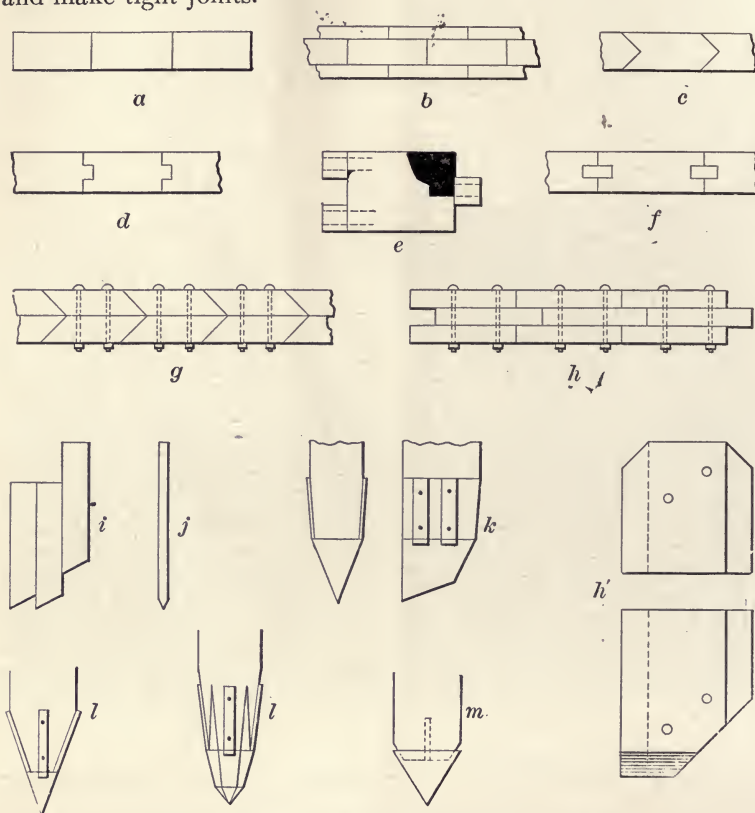


FIG. 38.—SHEET-PILES AND SHEET-PILE DETAILS.

Ordinary tongue-and-groove piling (*d*) is frequently used, but a more frequent form is that shown at (*e*), like that used on the Robinson circular dam. The two pieces forming the groove and the piece for the tongue are spiked to the 9×12 with 6-inch spikes sloping upward. A sheet-pile dam on another pier of the Arthur Kill Bridge employed piling in which the grooves were made by making two saw cuts and cleaning out between

with a chisel, the tongue being formed in the same manner as at (j), the tongue being spiked in one side.

A method which is not often employed is shown at (j), two



FIG. 39.—CHARLESTOWN BRIDGE. DRIVING WAKEFIELD SHEET-PIILING.

grooves being made in the sheet-pile and a key driven after the piles are down. Should the piles not drive in perfect line, and the groove fail to match, the method will not be found to be a success.

Sheet-piling formed of two or more plank bolted together

is being extensively used, one of them (*g*) being formed by two plank sawed with beveled edges and bolted together to form a pile similar to (*c*). This forms a pile which will drive easily on account of having some size and which will require fewer supports in the shape of waling-pieces.

Several examples already given describe the use of Wakefield patent sheet-piling (*h*), the method of sharpening being shown at (*h'*). This is constructed of three layers of plank from 1 to 4 inches thick, according to the pressure to be sustained. The center plank must be sized to keep the tongue and groove uniform, and the plank are bolted together with six bolts for a length of from 16 to 20 feet, two bolts near each end and two intermediate. For long piles, spikes should be driven between the bolts. The bolts vary from $\frac{3}{8}$ inch for 1-inch plank to $\frac{3}{4}$ inch for 4-inch plank. A coffer-dam constructed with this piling is shown in process of construction in Fig. 39, for the foundations of Charlestown Bridge near Boston. A description of this will be given in the next article.

Pile-shoes for use on round or square piles are shown at (*l*) and (*m*), (*l*) being patent forms. Straps of bar iron are used in many cases with success, for main piles, and sheet iron of $\frac{1}{8}$ inch thickness, bent to a "V" and spiked on, is often all that is necessary when shoes must be used on sheet-piles.

The thickness of sheet-piling should be sufficient to prevent the plank from bulging and should be calculated to stand a water pressure due to the depth, and for a span equal to the distance between the waling-timbers or other supports. This would necessitate wales every 6 feet for 3-inch plank under 5-feet head, or wales every 3 feet for a 21-feet head. Plank $4\frac{1}{2}$ inches thick would require wales every 7 feet under a 9-feet head, or every 5 feet for an 18-feet head. Timbers 9 inches thick will carry 9 feet under a 20-feet head, while the 15-inch timbers of the Buda Pesth dam would carry 12 feet under a 33-feet head.

Good timber should always be employed if it can be procured, or, if faulty stuff must be used, allowance must be made by using thicker piles and by placing the wales closer together.

CHAPTER V.

CONSTRUCTION WITH SHEET-PILES.*

WATER pressure against the sides of a sheet-pile coffer-dam is seldom provided for in an accurate manner, the thickness of the piling being usually decided upon from past experience, as are also the size and spacing of the guide-piles and wales.

These are points where guess-work should be eliminated, as otherwise good coffer-dams are often seen where the pressure has so bulged the plank as to cause leakage. While this may perhaps be corrected by additional bracing, simple calculations may easily be made to determine the size beforehand.

The pressure against a coffer-dam may act as at (a), Fig. 40, the sheet-piling being in the condition of a beam fixed at one end and loaded with a gradually increasing weight, as shown by the dotted lines, due to the pressure of water or puddle at 62.4 pounds per cubic foot. Then the load on a width w of the wall is $124.8wd^2$ and the moment of the pressure is $83.2wd^3$. Taking the allowable unit stress on wet timber at 1,000 pounds per square inch, the thickness t of the sheet-piling may be obtained from the formula

$$t = \sqrt{.496d^3},$$

in which d is to be taken in feet, and the resulting value of t will be the thickness in inches of the sheet-piling.

* The assumption that the pressure of puddle will be the same as water pressure is made advisedly. It is true that *very* wet clay, approaching a fluid condition, will exert a much greater pressure, but it would then be useless as puddle. Dry clay would exert a pressure of less than half that due to water, so it has been assumed that *wet* clay or puddle would exert the same force as water. Should it exceed it for a short time no damage would be done, owing to the low unit stress adopted.

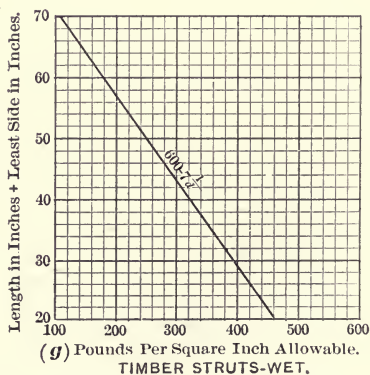
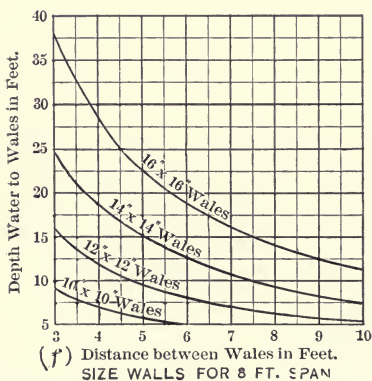
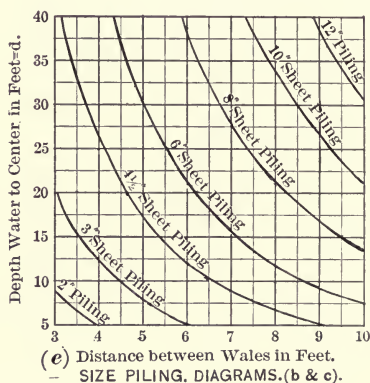
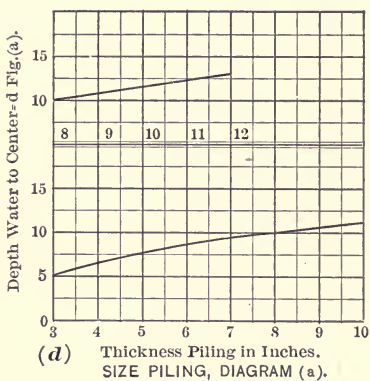
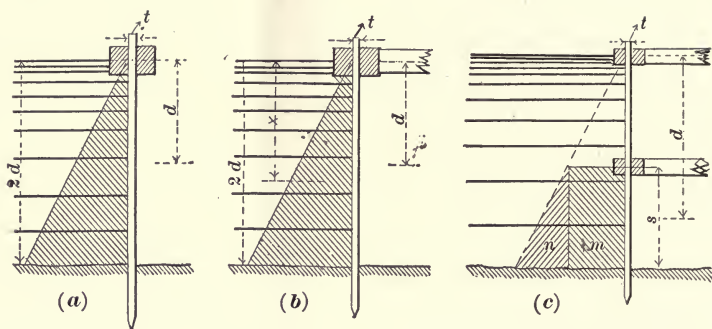


FIG. 40.—ARRANGEMENT AND DIAGRAMS OF SIZES FOR SHEET-PILE COFFER-DAMS.

This formula has been expressed in a graphic manner in diagram (d), Fig. 40, from which, knowing the depth of water $2d$, the thickness of piling may be read directly without calculation.

The addition of a strut, as at (b), Fig. 40, places the sheet-piling in the condition of a beam supported at the upper end and fixed at the lower end, but for practical reasons it is best to consider it as merely supported at both ends. The load will be the same as in the former case, $124.8wd^2$, but the maximum moment will occur at a point x which is a distance from the top equal to 1.16 times d , and has a value of $32wd^3$. The thickness t may be found from the formula

$$t = \sqrt{.192d^3}.$$

When the section of the plank to be calculated is located as "s" in (c) of Fig. 40, it is in the condition of a beam fixed at both ends and loaded with a uniform load m and a triangular load n . The exact analysis of this is too lengthy to be taken up here, and reference may be made to page 195 of Wood's "Resistance of Materials."

For practical purposes we may consider the load as all uniform and due to the head acting at the middle of the span. This will give a load of $62.4wds$ on the span s for a width w , and a moment of $7.8ds^2$, which gives a formula for practical use, for a unit stress of 1,000 pounds per square inch of

$$t = \sqrt{.047ds^2}.$$

This is closely represented graphically in diagram (e) of Fig. 40, which may also be used for case (b) by taking the depth of water to the middle of the span. For example, when the depth of water to the middle of the span is 15 feet, find this in the vertical column to the left, and if 6-inch sheet-piles are to be used, follow the horizontal through 15 feet until it intersects the 6-inch curve and vertically beneath will be found the maximum spacing of wales, 7 feet 3 inches.

The size and spacing of wales may be taken from a similar

diagram (f) of Fig. 40, which assumes the guide-piles to be 8 feet apart. The spacing of struts or braces will vary so much that the load must be calculated, and when this and the length are known the size may be calculated from diagram (g) of Fig. 40, which is for wet timber.

From the formula

$$p = 600 - 7(l \div d),$$

in which p is the allowable stress in pounds per square inch, l is the unsupported length in inches, and d the least side of the stick in inches.

Where two rows of sheet-piling are to be driven to form a puddle-chamber, if they are to be efficiently braced from the inside of the coffer-dam, it will be sufficient to have a thickness of puddle of from 2 to 4 feet to exclude the water, depending on the quality of the puddle. Where there is to be no internal bracing, but two rows of sheet-piling braced together and filled with puddle are to resist overturning, the common rule is to make the width of the puddle-chamber equal to the height above ground, up to 10 feet. When the height exceeds 10 feet, add one-third to the excess height to 10 feet for the width.

When the puddle-chamber becomes very wide it is often divided into several compartments, as was shown in Fig. 5, and stepped in a similar manner. When the bottom is rock overlaid with a thin deposit of clay or gravel, the sheet-piles may be driven around an open crib-work for support, as was done at Harper's Ferry, on the B. & O. R.R.

Where the guide-piles are to be used, the waling-pieces are framed in, as was specified on the Hutcheson Bridge, as shown at (a), Fig. 41, where the guide-piles are of sawed timber. The wales are spaced slightly farther apart than the thickness of the sheet-piles, to allow clearance in driving, the space between the guide-piles being filled out with a key pile to fill the panel tightly. This method is but little used with tight piling, that shown at (b), Fig. 41, allowing the piling to be driven continuously, by removing the spacing blocks as they are reached, and substituting bolts

through the sheet-piles, firmly connecting the piles and wales together.

A very satisfactory method is described in the *Engineering News* of May 12, 1892, which was used by A. F. Walker. Having occasion to do a large amount of work it was desirable not to go to the expense of squared guide-piles. Round guide-piles (*P*) were first driven 7 feet apart, and cut off to a level. Caps were then drift-bolted to the tops, allowing them to project slightly beyond the face of the round piles, thus forming a permanent

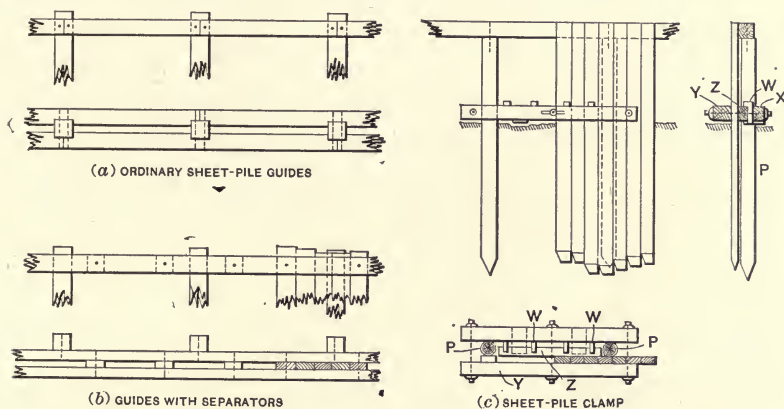


FIG. 41.—SHEET-PILE GUIDES AND CLAMPS.

support for the top of the sheet-piles. Near the ground line was placed the clamp, consisting of two sticks (*X*) and (*Y*), connected by three bolts and drawn together as tight as the intervening piles or pile and gauge-block (*G*) will permit. The stick (*Z*) is then forced forward by the wedges (*W*) until the space between (*Z*) and (*Y*) is the same as the thickness of the piles. The pieces (*X*), (*Y*), (*Z*) are slotted for the middle bolt, and this permits of some adjustment. When one of the piles partially closes this slot, a notch is cut in the same large enough to receive the bolt, and the bolt is then slipped up to it and tightened. This allows of the next pile being driven as close as the others. When one panel has been completed the nuts are removed and the clamps moved forward to the next one, a notch being cut

in the end pile to receive the end bolt of the clamp. The piles are sharpened flatwise with a little more slope on the side facing the guide-piles, giving them a tendency to drive away from the guide-pile at the foot and bear against the cap at the top. A slight bevel is also given to the edge to make the foot crowd the adjoining pile. During the first half of the driving, the joint is held a little open at the top, but during the latter half, pressure is brought to crowd it toward its neighbor, and the joint will close as tightly as possible.

The use of single pieces of timber as wales, against which the sheet-piling is driven, is illustrated in the use of method (b) of Fig. 38, by Benj. Douglas, bridge engineer of the Michigan Central Railway. The coffer-dam (Fig. 42) was built without guide-piles, the wales being 12×12-inch timber bolted against the outside of the sheet-piling, by the brace rods 1 inch in diameter. The wales are held in place vertically by bracing of 2×12-inch pine plank, which are spiked on as verticals and diagonals to form a truss and also to stiffen the framework in general.

The sheet-piling is 6×12, and after being driven into the hard gravel bottom, the cracks were lapped by 1-inch boards. The bottom was uneven and accounts for the difference in height, the excavation at the high end being dumped outside at the low end, to assist in making the dam tight. The puddle-chamber was 2 feet 8 inches wide and was filled with clayey gravel. The plan also shows the grillage in place for receiving the foundation courses of the stonework. This is formed by 12×12 timber crossed, and drift-bolted together with 1-inch round and 18-inch long drift-bolts.

The account of the Arthur Kill Bridge foundation in Vol. 27 of the "Transactions of the American Society of Civil Engineers," by A. P. Boller, consulting engineer, covers a very interesting experience with sheet-piling on pier No. 5: "This pier is near the edge of the marsh forming the Staten Island shore, which is barely flooded at extreme high tides. Borings indicated about 30 feet from the surface to hard bottom, consisting of mud, mud and clay, clay and shale to the bottom of shaley clay, in which the pier was

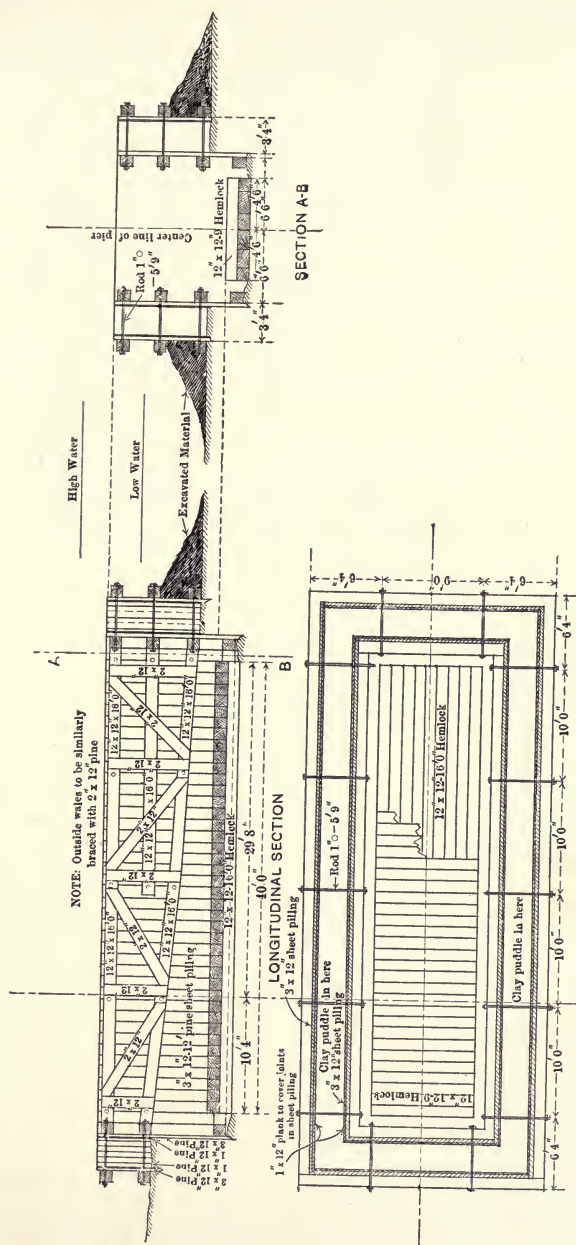


FIG. 42.—COFFER-DAM FOR ANN ARBOR BRIDGE. MICHIGAN CENTRAL RAILWAY.

to be founded. Experience on other work of a similar character indicated that the founding of this pier would be accomplished with little difficulty. The area of the foundations was inclosed with a tongued-and-grooved sheet-pile dam of 4-inch yellow pine plank. But it was found impossible to hold the plank at a depth of 15 feet, the mud and clay becoming puddled with water, and despite all efforts at bracing, the plank shoved inward to such an extent as to spoil the whole dam before we were half way down. A second dam was therefore driven around the first one, but this time with 10×12-inch tongued-and-grooved timbers, in one length to reach the extreme bottom. These timbers were grooved by slitting the grooves out at the mill with a circular saw, and chiseling the blank so formed free. The tongue was an independent spline, $2\frac{1}{2} \times 4$ inches, of dry wood and nailed in one groove. The timbers were shaped at the feet to drive close. This dam was hard driving, but was finally accomplished, when digging was resumed and the old dam removed piecemeal as we could get in the braces. The bottom was reached within a perfect dam, with only one bad leak in the northwest corner, due to the shattering of a small piece of one tongue during the driving. As it was impossible to stop this leak from the inside, and the outside was inaccessible, to prevent washing the concrete, the leak was led off in a box at the side of the dam to the sump-well, and the footing course of concrete, filling the whole area of the dam about seven feet deep, was gotten in place."

This example emphasizes in a very decided manner many of the statements that have been made heretofore. While no doubt the removal of the old dam was attended with much expense, its inclosure entirely within the new sheet-piling rendered the prosecution of the work comparatively certain.

An example of the driving of sheet-piling on a slant, to prevent crowding in at the bottom, is shown in Fig. 43, which is a cross-section of a sewer coffer-dam used on the Metropolitan Sewerage Systems of Massachusetts by Howard A. Carson, chief engineer, and described in the *Engineering News* of Feb. 8, 1894.

The outlet into the ocean at Deer Island begins at a point about

60 feet inside the high-water line, and about 1,850 lineal feet is from 5 to 10 feet below high water. This necessitated the coffer-dam, which was constructed with bents every 6 feet and with 2-inch plank inside the high water-line, but for the remaining distance of 4-inch matched plank. The excavation was done by means of buckets, traveling-derricks, and dump-cars, the latter being emptied at the sides and ends of the trench. The leakage from the

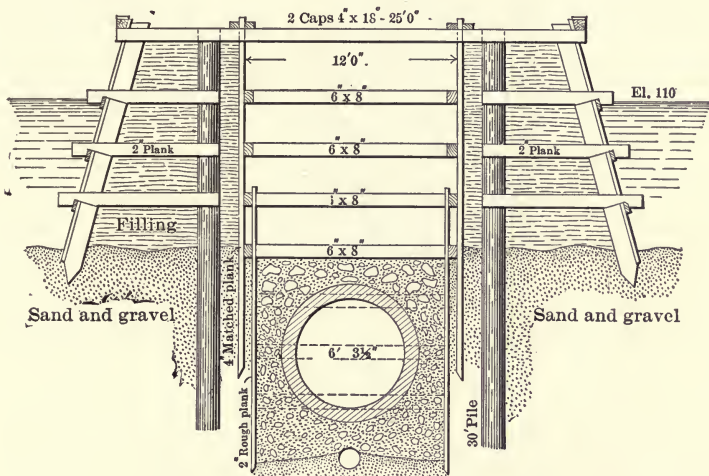


FIG. 43.—SEWER COFFER-DAM. BOSTON SEWERAGE SYSTEM.

ocean was kept out by using centrifugal pumps, which pumped a maximum of 46,000 gallons per hour. The concrete, which has large boulders imbedded in its surface the size of paving-stones, was carried up to the level of the ocean bottom.

From the middle of June, 1893, when the work was begun, to the end of September, 526 feet of trench were completed. The size of the trench was 14 feet average depth and 10.8 feet average width, which made the excavation average 5.6 yards per lineal foot. The cost for the trench, including coffer-dam, sheeting left in, and back-filling, was \$44.00 per lineal foot.

Casual mention has been made in several places of the use of Wakefield sheet-piling which was illustrated at *h* and *h'* of Fig. 38 and which is further shown in Fig. 44. View No. 1 is of a

corner which is formed as in the plan No 2, a tongue being bolted on the side of a pile, when the corner is reached as in No. 3.

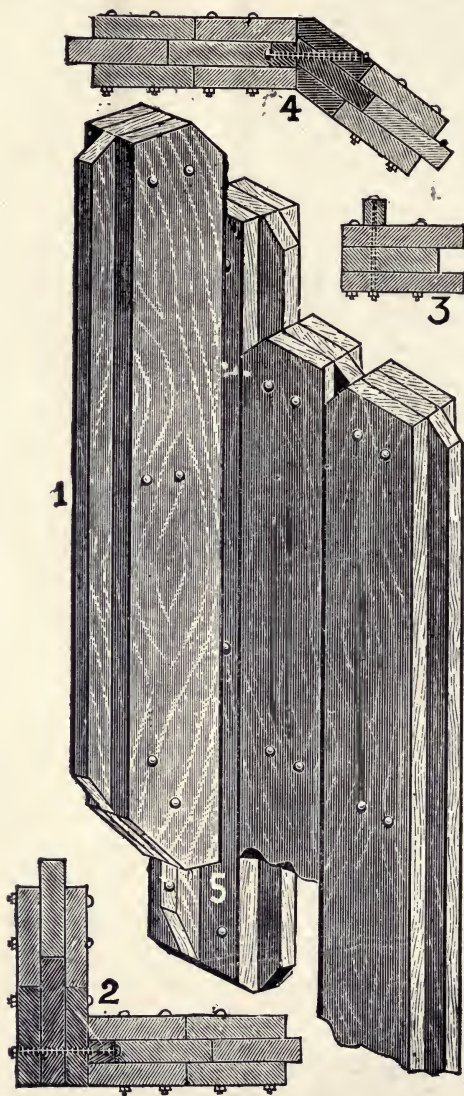


FIG. 44.—WAKEFIELD SHEET-PILING.

Any angle is turned by a similar method, which is shown by No. 4, or the piles may be driven to form a curve. The essential fea-

tures of the system are the triple lap or long tongue and groove which excludes the water, and the use of ordinary plank, which can be easily obtained. The center plank should be sized to a uniform thickness, to insure the tongues fitting the grooves, and to make driving easy, while the three plank are to be bolted and spiked together to cause them to act as a compound beam and not as separate plank like the system of (b), Fig. 38. It is recommended to use a $2\frac{1}{2}$ -inch tongue on 1-inch boards and $\frac{3}{8}$ inch bolts. For $1\frac{1}{2}$ -inch plank a 3-inch tongue, for 2-inch and $2\frac{1}{2}$ -inch plank a $3\frac{1}{2}$ -inch tongue and $\frac{1}{2}$ -inch bolts, while for 3-inch plank a $3\frac{1}{2}$ -inch tongue and $\frac{5}{8}$ -inch bolts are to be used, and the same size bolts for 4-inch plank, but a 4-inch tongue. Two bolts are to be staggered in every 5 to 8 feet of the length of the pile, and spikes used between the bolts on long piles.

The La Grange lock on the Illinois River was inclosed with this piling, under the direction of Major W. L. Marshall, Corps of Engineers. It was intended to back the sheeting with earth, but as both dredges broke down the water-tightness was entirely dependent on the Wakefield piling, and under a 7-feet head no leaks were developed. The piles were made of three plank 3×12 inches by 22 feet long and with a 3-inch tongue; they were driven by three pile-drivers with hammers of from 2,800 to 3,000 pounds through sand and mud, and in one place a layer of shells. There was no difficulty experienced in driving the piles without special appliances.

The use of 1-inch boards in this form (Fig. 45) is described by H. F. Baldwin, chief engineer of the C. & E. I. Railway: "In constructing our second track over the Kankakee River at Momence, Ill., it was necessary to extend the piers in that river. The bottom is limestone and the surface is very irregular. We tried several days and finally succeeded in constructing a cofferdam with two parallel walls of sheet-piling. We then tried the Wakefield triple lap-piling, constructed of 1-inch boards sharpened to an edge, $2\frac{1}{2}$ tongue and groove, which were driven with sledges until the piles, which were soft pine, conformed to the uneven surface of the rock. This piling was driven around

cribs loaded with stone, and, after the piling was driven, gravel was put outside the coffer-dam, after which no trouble was experienced in pumping out the water."

The work on the foundations of the new B. & O. R.R. bridge over the Potomac River at Harper's Ferry was similar in many respects to the above, and the system was found to be very satisfactory.

References were made to the use of this piling on the Charlestown Bridge at Boston and the driving of the piles shown in Fig.

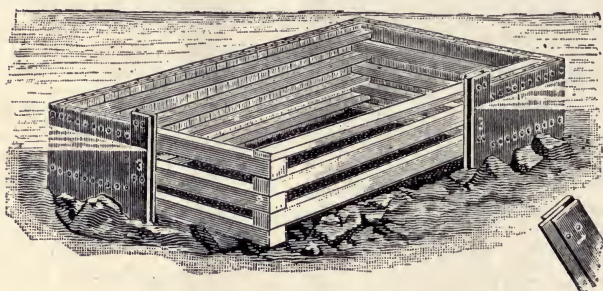


FIG. 45.—TYPE OF MOMENCE AND HARPER'S FERRY COFFER-DAMS.

39 The work was under the charge of Jno. E. Cheney, consulting engineer, and was successfully carried out. The piling were driven principally as forms for concrete foundations, and but little care was taken to make the dams water-tight. After the concrete was deposited they were used as coffer dams against a 6- or 7-feet head of water. They were 18 feet 6 inches by 119 feet (Fig. 46) and in some cases were 30 feet below low water or 40 feet below mean high water. The piling was made of 2-inch plank and driven with an ordinary pile-driver. The pumping was done with a 20-inch centrifugal pump, and in some cases a 12-inch Follansbee pump of the propeller type was used.

The construction of the sewerage system at Fort Monroe, Va., under Capt. Thos L. Casey, Corps of Engineers, is described in the report of the Chief of Engineers of 1896. The work was done on the general plans of Rudolph Hering, consulting sanitary engineer. One of the special difficulties en-

countered "was the building of a sewage tank 50 feet in diameter, with walls of brick 2 feet in thickness, exteriorly diminishing to 3 feet at the center, the inferior reference of which was 20 feet below low water. As described in the report referred to, this was accomplished very successfully by excavating a large area to the reference of ground-water, some 5 or 6 feet below the surface,



FIG. 46.—COFFER-DAM ON CHARLESTOWN BRIDGE.

and then driving by the pile-driver and water-jet combined, two concentric twelve-sided polygons of Wakefield sheet piling 28 feet in length, 30 and 22 feet from the center, about the circumference of the shallow excavation. (Fig. 47.) The material, consisting of fine water-soaked sand, with a small admixture of clayey matter and fine gravel, was then excavated between the polygons to a reference of 20 feet, transverse shoring braces bearing upon stout stringers being put in at intervals as the work proceeded. The material did not vary much in its general nature, but a number of old piles were taken up, some of which

did considerable injury to the sheet-piling when driven, as shown in the subsequent excavation. The water was controlled by a powerful steam-pump having its point of suction fixed, the water being permitted to flow toward it throughout the circumference. It was noticed that ground-water came through the sheeting very freely at first, but that it constantly ceased to flow to any great

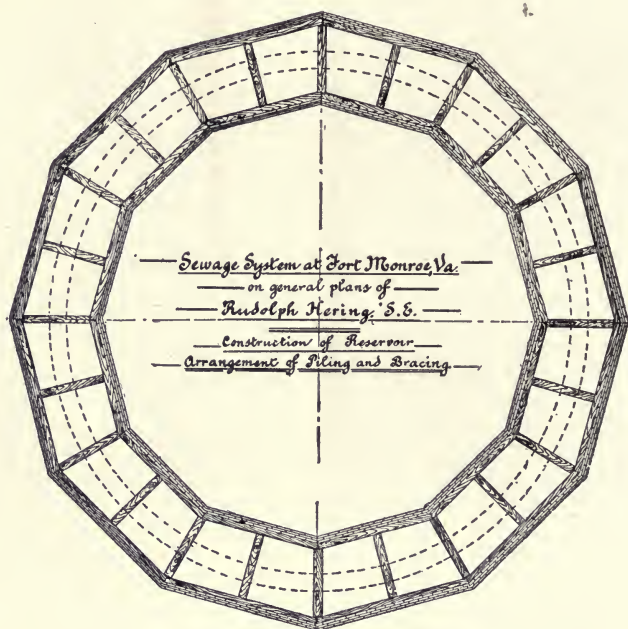


FIG. 47.—RESERVOIR COFFER-DAM. FORT MONROE, VA.

extent at a height of a few feet above a point of excavation as this continually descended, owing to the rapid drainage of the strata. The interior core, in fact, became quite dry, so that in excavating after the walls were laid, no water was encountered until the bottom of the external concrete ring had been virtually laid bare. Upon attaining the reference—20 feet, the excavation ceased and hand-mixed concrete was deposited directly upon the bottom, as this was considered to be sufficiently firm, the pump being stopped temporarily in order to prevent a flow. The concrete was rammed firmly against the outer sheeting externally and

against plank forms with triangular cross-section resting against the inner sheeting internally, until 6 feet in depth had been put in place. The portion of the ring at the pump suction was filled rapidly with concrete in bags. The 2-foot brick wall was then carried up from the axial line of the concrete ring, the space between the wall and the outer sheeting filled with sand, except about 6 inches at the base of the wall, which was of concrete. The braces were removed as successively attained, the inner prism of dry sand being held securely by the sheeting and the extreme top struts, which were left in place until the inner core was completely excavated. On the completion of the latter work to reference—20 feet, the water which came in freely from without under the concrete ring at several points was conducted in a peripheral trench to the fixed point of pumping. No water came upward and the middle portions of the bottom became perfectly dry. The inner sheeting was cut off at the base of the ring, boards were placed transversely over the peripheral trench, a duck tarpaulin coated with hot asphalt laid down, and concrete rammed in place until the concave bottom with sump channel had been completed, leaving only the pipe, through which the ground-water had been pumped continually, night and day at about 1,000 gallons per minute, penetrating the concrete. In order to fill this pipe, it was cut off above the level of permanent ground-water, and after the water within had attained the level of ground-water in the surrounding area and had become perfectly quiescent, neat cement in paper bags was dropped within, being retained at the bottom by the closed valve; the bags were readily broken up by a long pole thrust down the pipe. The latter was then cut off at the level of the bottom and a coating of cement plaster applied throughout. The resultant leakage through the bottom did not exceed about a gallon a minute and this will be greatly reduced by the infiltration of sand from beneath."

Further illustrations of the use of sheet-pile coffer-dams will be given; then the operations of dredging, pumping, and concreting described at some length.

CHAPTER VI.

CONSTRUCTION WITH SHEET-PILES (CONTINUED).

VARIOUS combinations of the sheet-piling shown in Fig. 38 may be made, when occasion demands, or modifications may be made that will perhaps render the available material more effective. For example, the form (g) may be modified to the form shown in Fig. 48, which has the advantage of a wider lap, and

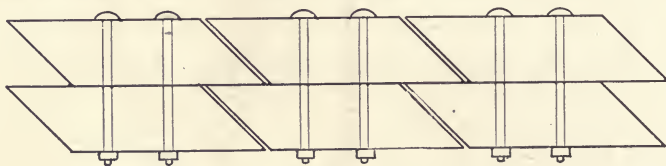


FIG. 48.—COMPOUND SHEET-PILE.

should the piles not draw tight together in driving, no crack will be left open to admit the water. Then the piles of this form will act as guides to the ones being driven, similar to the ordinary tongue-and-groove piling. Other combinations and arrangements will readily suggest themselves as necessity may demand.

The use of sheet-piling is often accompanied by a great deal of trouble in producing tightness, and as a matter of precaution, the very best method possible should be adopted in making the piling.

The coffer-dams constructed at Chattanooga for the Walnut street bridge over the Tennessee River, under Edwin Thacher, consulting engineer, were described in the *Engineering News* of May 16, 1891.

Four piers were founded by this method, but the account of

pier No. 2 will fully illustrate the work. The bed-rock, which was level, was covered by cemented sand, gravel, and boulders, of which 320 yards were removed. The coffer-dam was built 18 feet high, or 8 feet above low water, to provide for a future rise. The inside was made large enough to allow of a space of 4 feet all around the base of the pier, and the space between the sheet-piles for a puddle-chamber was made 9 feet. This was filled to an average of 12 feet with a clay puddle, of which there was 900 yards used. As a protection, there was placed outside the dam about 450 yards of puddle, and a breakwater was built up-stream. About 38,000 feet of timber were used in the dam and breakwater.

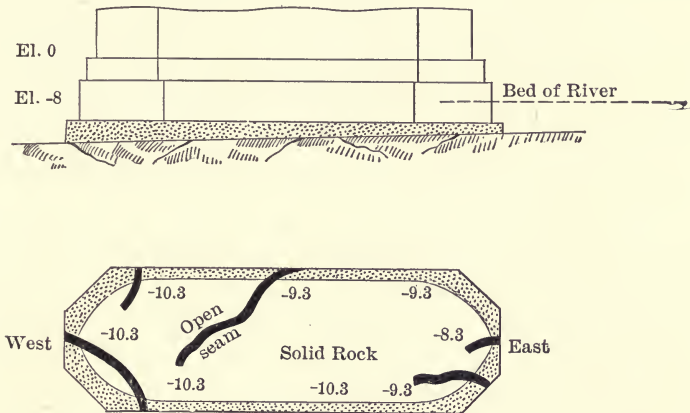


FIG. 49.—CHATTANOOGA BRIDGE, BED-ROCK PIER No 3.

After the dam was completed a rise of 30 feet washed out about half the puddle, and one end was crushed by a raft, but the repairs were made without serious trouble. No extra amount of pumping was required on any of this work except pier No. 3, where the seams in the bed-rock required pumps with a capacity of 5,000 gallons per minute, and these did not suffice to keep the water down, until the seams were closed by laying sacks of concrete over them and weighting them down with large stones. The location of these seams is shown in Fig. 49.

The framework and wales for a sheet-pile coffer-dam, used in founding the pier for the Baltimore street bridge at Cumber-

land, Md., are shown in Fig. 50, and this was described in the *Engineering News* of July 21, 1892, by H. P. Le Fevre, engineer in charge. The frame was built in place on two canal-boats and after completion was suspended from the old Bollman truss which the new bridge replaced.

The depth of the water was 4 feet, and about 6 feet of very loose quicksand and small round pebbles overlaid the hard bottom.

After the boats were removed, the frame was lowered to its place, the sheet-piling driven, and the dam pumped out with a 6-inch pump. The foundation was laid on the hard bottom under the quicksand, after this had been removed.

The grillage was made of two courses of 15×15-inch clear white oak, around which was built a framework, and the open spaces of the grillage were then filled with a concrete made up of one part of Cedar Cliff cement to two parts of sand and four parts of hydraulic limestone broken to pass through a 2-inch ring. Upon this were laid the footing courses of the masonry.

Another ordinary sheet-pile coffer-dam which gave good satisfaction was used at the Sandy Lake dam on the Mississippi River, by Major W. A. Jones, Corps of Engineers, and as the account contains so much of value it will be quoted in full from the 1894 report of the Chief of Engineers:

"The coffer-dam is composed of two rows of round piles, 12 feet from center to center of piles, with the exception of 62 feet of the east end of the upper part, where they were driven 14 feet from center. The piles in each row are 8½ feet from center to center, cut off at an elevation of 1,217 feet above sea-level and capped with 12×12-inch timber. The inside row of sheeting is 4×12-inch, and the outside 6×12-inch plank. The sheeting is cut off at an elevation of 1,218 feet above sea-level, or 2 feet below the flowage line. One-inch rods of round iron, 8½ feet apart, pass through the caps to prevent the filling from spreading the two lines of sheeting at the top.

In May, 1892, when a flood occurred, the outside of the coffer-dam was raised 3 feet by splicing 3-inch planks to the outside row

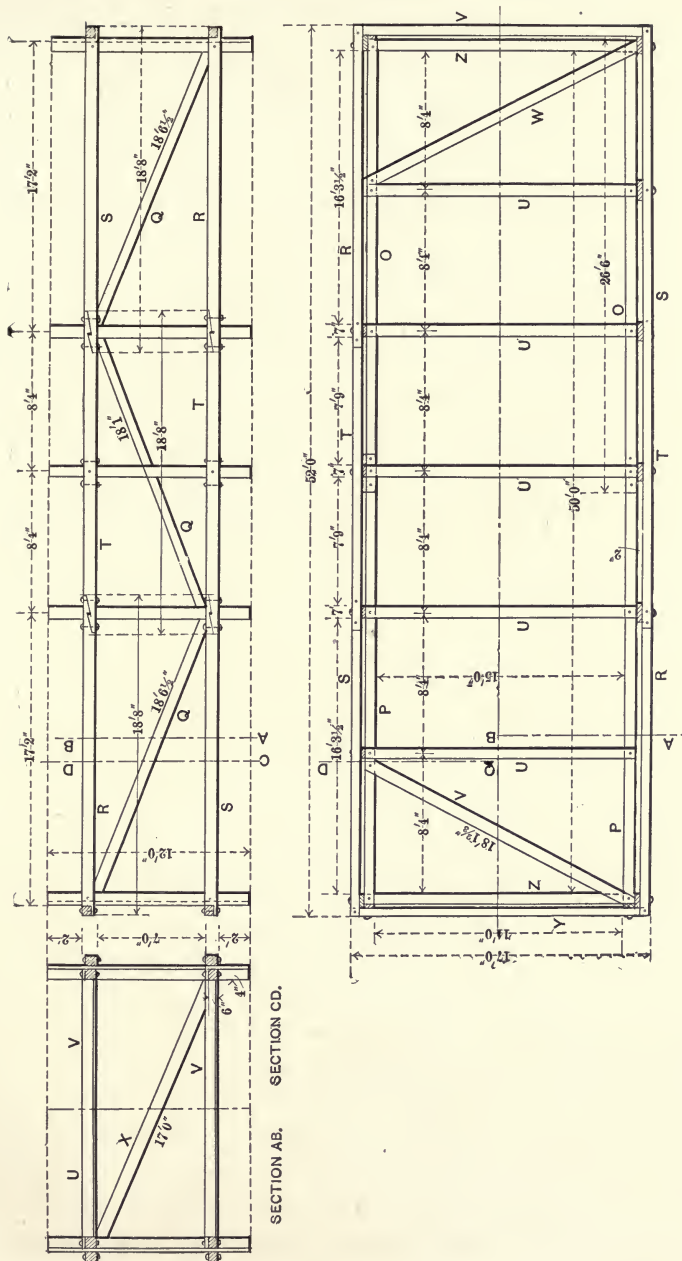


FIG. 50.—FRAMEWORK OF COFFER-DAM, CUMBERLAND, MD.

of sheeting and then filling the triangular prism thus formed with earth. The cross-section of Fig. 51 gives an idea of the dam above the bottom, while the longitudinal section shows the framing down to where it rests on the bottom, the frames being joined by the 1-inch lateral rods of iron.

The total length of the coffer-dam is 829 feet, of which 742 feet is like that shown in cross-section and the other 87 feet like that shown in the longitudinal section.

The number of round piles driven in the foundation is 1,605. The driving was commenced on November 12, 1891, and completed on August 21, 1893.

The material in the foundation is sand, excepting in the

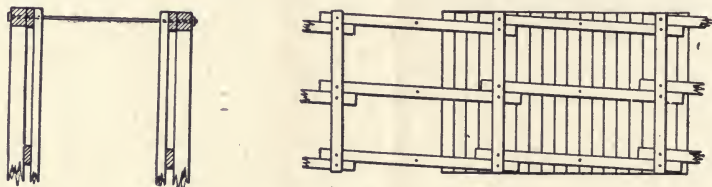


FIG. 51.—SANDY LAKE COFFER-DAM.

lower right-hand corner, where there is some blue clay overlying the sand. The sand in the foundation is not as compact as it is usually found in the bed of streams. In the south half of the dam, the surface settled from 6 to 4 inches during the driving. As the surface was settling, the driving became harder all the time. In the north half, which embraces the navigable pass, there was some settlement, but it was not as noticeable as in the south half. The surface had probably settled by the jarring of the hammers while the first half was being driven. The penetration of the piles is also greater than it usually is in sand foundations in the bed of streams.

The piles were all of Norway pine and well seasoned. Two Mundy steam hoisting-engines were used in driving, one a single-cylinder and the other a double-cylinder engine. In operating the hammer a $\frac{1}{2}$ -inch manila rope was attached to the pin connecting the lugs of the hammer, then passed over the sheave

at the top of the leaders, and next around the drum of the hoisting-engine.

When the hammer falls, it pulls the rope with it and unwinds it from the drum. This is what is termed driving with a 'slack line.' The blows are more rapid and keep the material around the piles looser than it would be in the case of using nippers. Iron rings of $\frac{5}{8}'' \times 2\frac{1}{2}''$ Norway iron were used to protect the head of the pile.

It is a well-known act in pile-driving that it is very important to keep the material from settling around the pile, once it has been loosened, until the pile is down; for when the material has settled, or even partially the penetration is diminished. The greatest load on a bearing pile is about $13\frac{1}{2}$ tons.

Sheet-piling was driven by a pile-driver, assisted by a jet of water from a steam force-pump. In driving all sheet-piles a cast-iron cap or follower was used which fitted over the head of the pile. On the upper side of the follower there is a wooden block of some seasoned or close-grained wood which receives the blow of the hammer. This device saves the head of the pile from being battered or splintered, and the pile can be driven to a greater depth than it could be without it.

In first using the jet on a sheet-pile, a groove was made in the inner edge to receive a $\frac{1}{2}$ -inch gas-pipe, which was connected to the force-pump by means of a $1\frac{1}{2}$ inch hose. The aperture at the lower end of the gas-pipe was reduced to a diameter of about $\frac{3}{8}$ inch. The water was thus forced to the bottom of the pile and the sand loosened.

This worked well until the sheet-pile struck gravel, when the nozzle of the pipe would become battered or filled with gravel. The pressure in the hose would then burst a coupling somewhere. Another source of trouble was the frequent breakages in the connection between the pipe and the hose, on account of the jarring of the hammer. This plan after a while was abandoned and the nozzle of the pipe was thrust by hand under the point of the pile. The piles are driven in the ground from 12 to 14 feet."

The construction of the Main street bridge at Little Rock, Ark., involved the construction of two coffer-dams, for piers No. 9 and No. 6. This work was done under the direction of Edwin Thacher, consulting engineer, whose original specifications called for pile foundations for these piers, the piles to be driven to bed-rock and cut off 4 feet below low water, to receive a grillage of 12×12-inch timbers to receive the masonry. The size of the grillage being 12 and 13 feet wide by 34¹/₂ feet long and resting on forty eight and sixty piles respectively, the piles being of good sound oak or pine at least 7 inches in size at the small end and not less than 12 inches at the butt when sawed off.

The coffer-dams were constructed, as can be seen from the view in Fig. 52, by driving guide-piles, to the top of which are drift-bolted square guide-timbers. The sheet-piling of 3-inch tongue-and-groove stuff was driven against the outside of this timber, and the excavation banked up against the outside. They gave excellent satisfaction and caused little trouble as the water was shallow.

The piers were constructed of Portland cement concrete, the facing of 2 inches thickness being a mortar of one part cement to two parts of sand, while the balance was of concrete of one part cement, three parts sand, and six parts of broken stone.

Where sheet-piles are to be driven on rock bottom or through earth or gravel to rock bottom, they should be driven hard enough to broom up and form a close joint with the rock. This has been accomplished also by driving the piles with a thin edge until they fit the rock bottom, when they are drawn and after cutting them to conform to the contour of the rock, they are redriven, thus forming a tight joint. This method while very good, is too expensive for general adoption.

Coffer-dams are quite frequently constructed for the repair or removal of existing piers. A pier which was constructed in 1840 in the river Farnitz, at Stettin, Germany, became an obstruction to navigation and it was decided to remove it. The work was described in the *Engineering News* of July 14, 1892.

Its exterior showed a facing of granite laid in hard Roman

cement, and soundings revealed the existence of a course of sheet-piling around the pier, with a protection of riprap at its foot. The original drawing of the pier showed a pile foundation. The



FIG. 52.—COFFER-DAM AND CONCRETE PIER, LITTLE ROCK, ARK.

specification prescribed the use of the old course of sheet-piling shown at *a*, Fig. 53, for the construction of the coffer-

dam. Owing to the belief that the existing sheet-piling, after having served such a length of time, would not be sound enough to permit of its use in the erection of a coffer-dam, local contractors could not be found and the work was let to an outside contractor.

The preliminary work was begun by picking up the riprap around the foot of the pier with a claw dredger mounted on a raft. Some of the stones weighed as much as a ton. The bottom of the river, after the riprap had been cleared away, was found to be covered with a layer of concrete, consisting of pieces of brick and cement. This was brought up in large slabs. The pier itself was found to be of rubble masonry, composed of irregular-shaped granite blocks with the interstices filled with brick, laid in cement mortar. The single stones were detached and swung off by the claws of the dredger. Their average weight was about $1\frac{1}{2}$ tons.

After the masonry had been pulled down to nearly the level of the water a row of sheet-piling, shown at *b* in Fig. 53, consisting of piles 7 inches thick, was driven to a depth of nearly 10 feet. The space between the old and new sheet-piling was filled with new clay. To keep the interior free from water two pumps were employed. After putting in the necessary bracing the work of removing the masonry to the bed of the river was continued. A shell of the latter, however, was left standing. Then the timber platform on which the masonry had been resting and the layer of concrete below were taken out, exposing a layer of clay underneath. While attempting to pull one of the foundation piles a stream of water rushed through the opening thus formed, so that this plan had to be given up and blasting resorted to. To do this the tops of the piles were bored to a depth of 13 feet and filled with 8.8 pounds of dynamite each. The initial charges consisted of 10.6 ounces in air-tight canisters. The shell of masonry left standing received four cubical charges of 8.8 pounds each. In all sixty-eight charges, consisting of 616 pounds of dynamite, were used. The electric current for the blast was divided into three currents, each being attached to

an induction apparatus. The blasting, however, did not prove to be as effective as was anticipated, owing to the dissolving action of the water, and several charges were taken out intact. The clearing away of the wreck was almost entirely done by

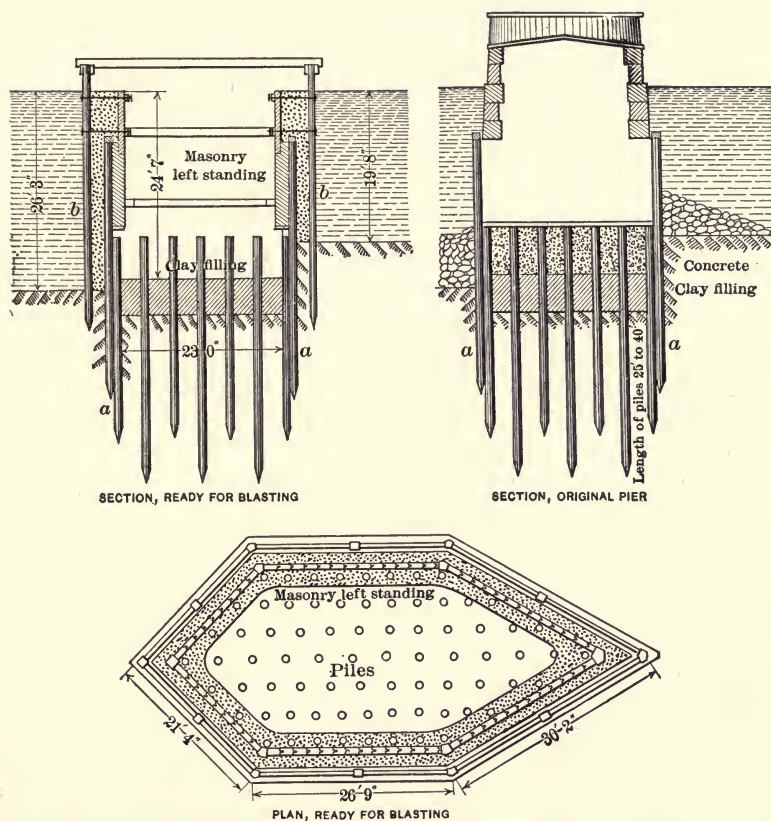


FIG. 53.—REMOVAL OF MASONRY PIER AT STETTIN, GERMANY.

the claw dredges. The piles, which were split and loosened in their sockets by the force of the explosion, were pulled up by windlasses mounted on flatboats. The work of removing the pier lasted nearly nine months and the cost was about \$8,700.

Another example of the removal of a pier was at Gadsden, Alabama, where a pivot pier in the Coosa River had tilted. The

pier had been built originally in a water-tight caisson and was supposed to have been founded on solid rock, but by some error a layer of gravel was left underneath and eventually the pier tilted down-stream 7 feet, nearly throwing the swing span into the river.

After the span had been blocked up to allow the passage of trains, a coffer-dam was built around the pier to give plenty of clearance to the old caisson. (Fig. 54.) This was constructed by driving three rows of sheet-piling through sand and gravel to bed-rock and puddling between them.

The sand and gravel over the rock was not removed from the bottom of the puddle-chamber before puddling and a great deal of trouble was experienced all through the work by leakage through the porous gravel. It is probable, too, that a poor joint was made between the sheet-piling and the rock.

Bents were erected upon the sides of the coffer-dam and by driving piles into the puddle and inside the dam, to carry a truss on each side of the span, which carried the drum and supported the main trusses at the center. When this had been tested by loading with trains of ore upon the bridge and found to be satisfactory, work was at once begun upon the removal of the old pier, by means of two fixed derricks on the false work and one floating derrick. The stones were marked as they were removed to insure their return to proper places when the pier was rebuilt, and were taken to the shore until needed again. When the masonry was all removed the grillage was broken up and taken out, after which the gravel inside the coffer-dam was cleaned out down to bed-rock. New footing courses were laid to take the place of the gravel and old grillage, and the old stonework relaid by placing each course in its former position as nearly as possible. The pier was about 80 feet high and contained about 1,100 yards of masonry. The work occupied from Sept. 15 to Dec. 25, 1888, and was done under the direction of Cecil Frazer. The description is taken from the *Engineering News* of April 13, 1893.

The construction of the piers for the Philadelphia and Reading

Railroad bridge over the Schuylkill, was accomplished by the use of a floating coffer-dam, the foundations being laid upon the bed-rock.

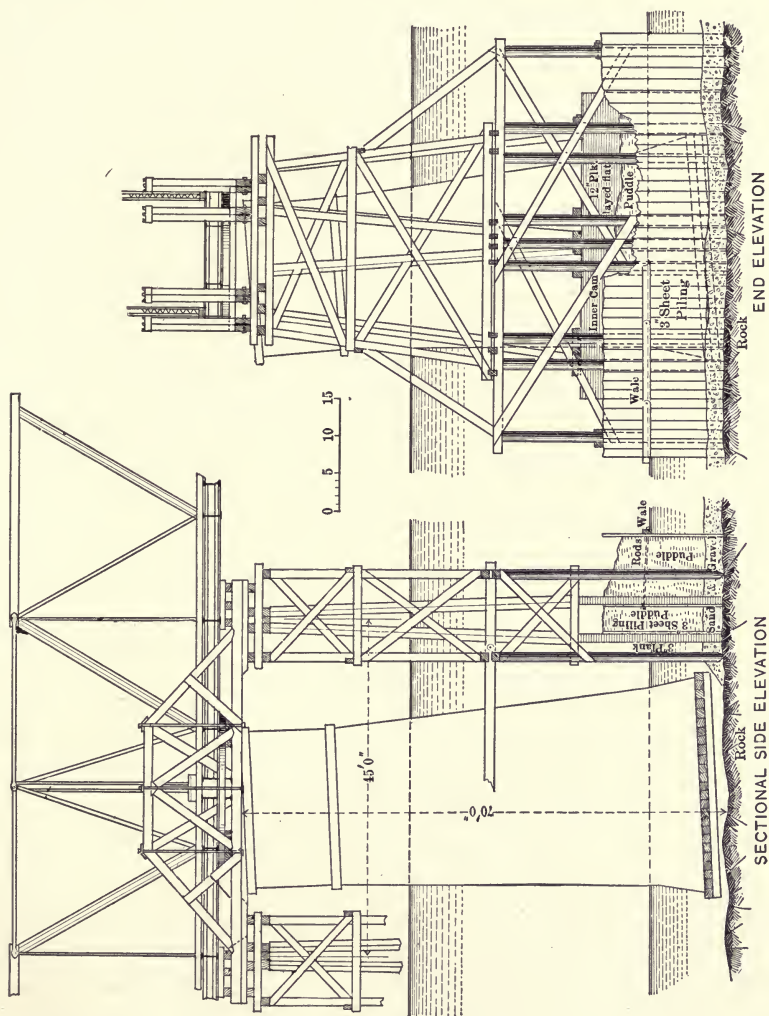


FIG. 54.—COOSA RIVER COFFER-DAM.

When in position for work the dam is rectangular in shape, 62 feet long, and 36 feet wide, outside dimensions, and 16 feet high. Each side consists of timber crib-work 10 feet wide,

making the inside dimensions $42' \times 16'$. At each corner there is a movable timber extending vertically from the bottom of the crib to some distance above the top. These timbers or spuds are shod with iron on the bottom, and serve to hold the dam in position while the sheet-piling is being driven. The dam is divided vertically through each short side into two equal parts, which can be floated separately to any desired position and afterwards joined together. Water-tight compartments are also used to hold stone when it is desired to sink the cribs.

When the two sections are united and placed in required position the spuds are dropped and the crib-work is sunk by letting water into the water-tight compartments, and putting in the necessary amount of stone.

Any irregularity in bearing between the bottom rock and the bottom of the crib is then corrected by a diver, who blocks up where required. Close sheet-piling of jointed plank 3 or 4 inches thick is then put on the outside and spiked to the cribs. Puddle, composed of clay and gravel, is then thrown around the bottom outside, and the dam is ready to be pumped out. When the masonry reached the height of the braces they were taken out and the dam was braced against the masonry.

The maximum depth of water encountered at Falls Bridge was 13 feet at ordinary water-level. Several freshets occurred during the progress of the work which did some damage to the dam. At one time, when a dam was ready to be pumped out, a rise in the river moved it down-stream about 30 feet, tearing off the sheet-piling. It was drawn back to place and successfully completed. To make a complete shift of the dam from one pier to the next, with a gang of six men, required about six or eight days, divided as follows: To take the dam apart and reset it, about three days; to sheet-pile, about two days; to puddle, about one day; and pumping out and puddling meanwhile required about one to two days, depending on the amount of the leakage. At each shift, a portion of the plank sheet-piling, perhaps 10 per cent., had to be replaced by new stuff. The pump used was located on a small steamboat, and was run by a steam-engine.

The amount of pumping required after the dam was once pumped out varied for the different piers; some dams required little pumping and others a good deal. Only one of the foundations required much leveling off of the river bed, and this one also gave considerable trouble to keep the water out, but the leaks were finally stopped by using gunny bags around them; the bags being drawn into the crevices by the force of the water, thus holding the puddle.

The floating dam was used for the three piers in the river channel, the two piers near the shore being put in with ordinary dams. The floating dam is still in good condition and could be used again if needed. The original dam, of which the one used at the Falls Bridge is an enlarged copy, was used for twenty-three or twenty-four settings.

The foregoing account is taken from the *Engineering News* of May 24, 1894, the description being by W. B. Riegner, who states also that the cost of the coffer-dam, including one set of sheet-piling, was \$3,000, while the total cost for five coffer-dams, including the two crib coffer-dams at the sides of the river, was \$14,000.

The subject of subaqueous foundations has been very fully treated in a series of lectures by W. R. Kinipple, M. Inst. C. E., before the Royal Engineers' Institute at Chatham, England.

The use of 6-inch pitch-pine close sheeting was made use of by him for a quay wall in the harbor of St. Helier, Jersey. They were driven to rock or as deep as possible with a 2,800-pound hammer, and the tops cut off a few feet beneath half-tide level, and clayey material banked up against the outside. The bottom through which the sheet-piles were driven was sand and clay.

The rock was laid bare to a depth of as much as 13 feet below low water and in sections which contained about 900 tons of water to be pumped out; this was done with a 16-inch centrifugal pump in about forty-two minutes.

Several leaks were developed under the piles, but they were promptly stopped by "stock ramming." The stock rammer which is shown in Fig. 55, is 3 inches in diameter, $3\frac{1}{2}$ feet long,

and banded top and bottom with iron. A $\frac{3}{4}$ -inch air-hole is bored up from its foot a distance of 20 to 30 inches, and covered on the bottom with a sole-leather flap, so that air is let in and suction prevented as it is withdrawn. The sheet-piles have $3\frac{1}{4}$ -inch holes bored through their sides, and cylinders of clay are inserted $3'' \times 9''$ long, similar to the work at Sault Ste. Marie. The stock rammer is inserted and driven by mauls as far as its length will permit when it is drawn out, and other charges inserted until no more clay can be driven. The hole in the pile being filled with a wooden plug.

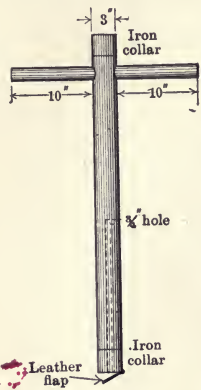


FIG. 55.—STOCK
RAMMER.

The piers for the Putney Bridge, over the Thames, were built by the same engineer, with single pile-dams to a great depth, by using 14-inch square piles, with elm-wood tongues, and driving them down through the mud and clay to the stiff clay bottom, so that practically water-tight work was secured.

In the construction of the docks at Victoria, British Columbia, he constructed a coffer-dam 500 feet in length, in a depth of 35 feet of water, the bottom being of rock and overlaid in places with sand and shells several feet in thickness. At the center the sand and shells overlaid a bed of clay.

Three rows of close 12×12 -inch sheet-piling were driven with two puddle-chambers of 7 feet each between. The guide-piles were 15×15 inches and the wales were 12×12 inches.

Where the dam rested on rock at the ends, heavy shoes were used on the piles and concrete deposited around their feet to make the work water-tight. This dam was completed in October, 1879, and remained thoroughly tight until the dock was completed over seven years later.

The arch bridge at Topeka, Kansas, over the Kaw River, which is being constructed on the Melan system, of concrete and steel, by Keepers and Thacher, the designing engineers, is a most interesting piece of work. The coffer-dams were required by the specifications to be water-tight, and to effect this 4×12 -inch

tongue-and-groove sheet-piling was used. The size of the coffer-dam for pier No. 4 was 18×55 feet in the clear (Fig. 56) and the piling was driven about 16 feet into the sand bottom or 22 feet below low water. The driving was done by a 1,600-pound hammer with 36 feet leads; the power being furnished by a 15 H.-P. hoisting-engine.

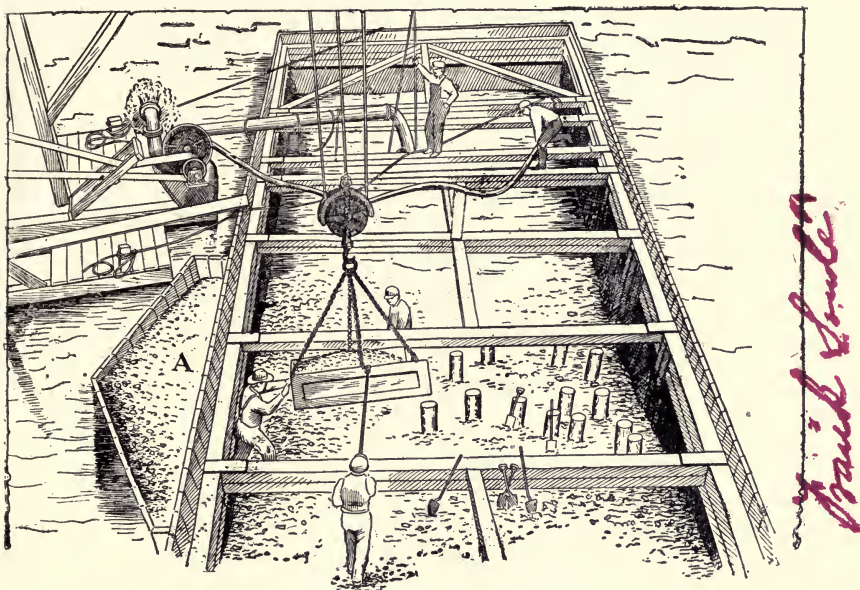


FIG. 56.—TOPEKA BRIDGE, COFFER-DAM No. 4.

"A" shows puddle to stop leak.

No puddle was used around the outside except to stop leaks, and the dam was kept clear of water with a No. 6 Special Van Wie sand pump. The capacity of the pump was 3,000 gallons per minute of water, and from 60 to 80 yards of sand per hour. It was operated with a 15-H.-P. engine. The other piers were handled in a similar manner and with no particular trouble.

The growing scarcity of timber will doubtless lead to the use of metal at some time in the future, to replace sheet-piling for coffer-dams, but where timber is abundant and reasonable care is exercised in its use, it will continue to be of great service in obtaining foundations by this method.

CHAPTER VII.

METAL CONSTRUCTION.*

THIN steel shells have been used extensively for foundation work, but in the majority of cases they have been retained as essential features of the permanent construction.

This is more particularly the case in locations where stone is scarce or expensive and it becomes necessary to substitute some other material for foundations. Tubular steel piers are constructed of two tubes, ranging from 24 inches to several feet in diameter, or, in the case of pivot piers, from 15 feet, with a single tube for a pier, to 30 feet or more.

In a number of instances the steel shells for ordinary piers have been made oblong, in the general form of a stone pier, and braced internally to hold them in shape during sinking, after which they are filled with concrete.

The metal shells for the Hawkesbury Bridge in Australia were of this character, 20 feet wide, 48 feet long, and with rounded ends. Each one was provided with three dredging-wells, each 8 feet in diameter, through which the dredges shown in the view (Fig. 57) were operated. While these piers were not used as coffer-dams, they were made water-tight by boiler-riveting, so

* Metal caissons have been used much more frequently in this country than have metal coffer-dams, the reasons being the cheapness of timber and its more easy application.

In England metal coffer-dams are more frequently used. The example given in this chapter—the Forth Bridge coffer-dams—might have been supplemented by reference to those used on the Clarence Bridge at Cardiff, the construction used being illustrated and described in *Engineering*, and is especially notable for the design of the bracing.

that by pumping water in and out the displacement could be kept constant, and in this way control the pier in an average tide of 5 feet. These piers were sunk, by dredging out the material from the inside, to the great depth of from 135 feet 8 inches to

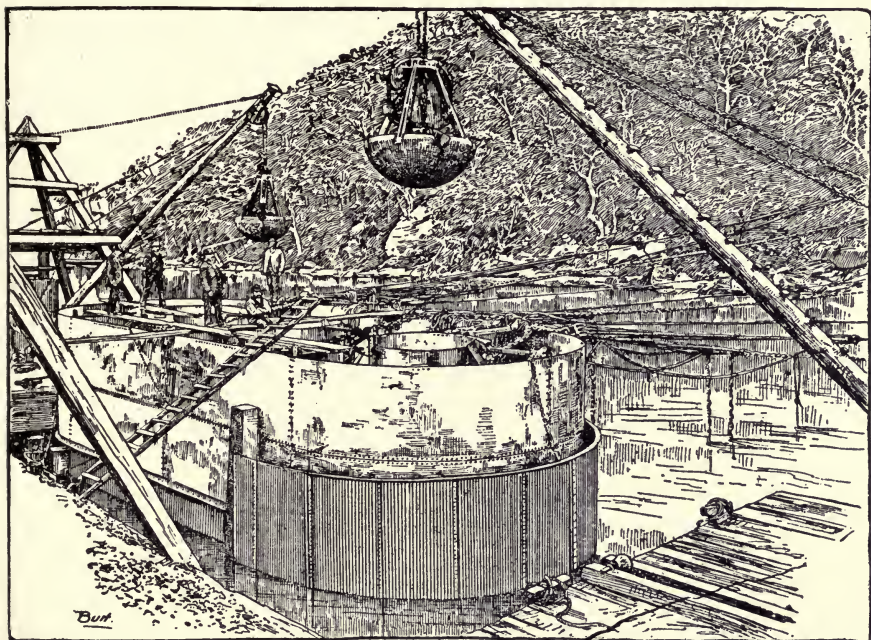


FIG. 57.—HAWKESBURY BRIDGE.

Caisson No. 6 in Process of Sinking, Showing Excavator and Shore Chains for Maintaining Vertical Position.

197 feet below the pier tops, or a distance of 155 feet below low water.

Both inclined and vertical cutting edges were used, with the result that the inclined ones were of frequent trouble and the vertical ones none whatever.

“If it is essential to increase the bearing surface at the bottom of the caisson to an area which is not required in the upper portion, this end can be secured by a vertical cutting-shoe of considerable height, with a step or steps into the smaller diameter. This is quite as efficient to secure the end in view as a long incline

on the cutting-shoe, and has decided advantages. In the denser material the vertical sides leave the ground undisturbed for some height close to the skin of the caisson, and a vertical guide is secured which is entirely wanting in the case of an inclined shoe. This guide is valuable in cases where the soil may differ in density under the shoe, and particularly so if the excavation has been carried too far below the bottom of the shoe. With an inclined shoe and a slip of soil into the dredging-well from one side more than another, experience in deep dredging has shown that there is a decidedly greater tendency to a horizontal movement than with a vertical shoe. The former has a flare to direct this sidewise motion in the first place, and nothing but a certain amount of disturbed material above the shoe to resist this tendency."

The above account is from the *Engineering News* of January 5, 1889, the work having been done under the direction of J. F. Anderson, of the firm of Anderson & Barr. The shells were filled with concrete up to low water, and masonry built from low water up to the top of the piers.

Such work may be made water-tight by riveting according to ordinary boiler-maker rules, or if extra thick plates are used this can be exceeded and the rivets spaced some farther apart. The joints may be made with ordinary laps and calked, or a very much better appearance may be obtained by the use of butt joints, and if desirable to avoid calking, then a calking strip may be used to make the joints tight. This is merely a cloth or canvas strip, thoroughly saturated with paint paste, and is laid between the metal surfaces, and the riveting draws the plates upon it and a tight joint will result. The shells will be filled with concrete as soon as the piers are in place and the foundation prepared, so that only a temporary use is required of the strip.

When metal cylinders are used simply as casings for concrete they need not be made water-tight, as they can be dredged out and have the concrete deposited through the water. The metal should never be less than $\frac{1}{4}$ inch in thickness, and on first-class work $\frac{5}{16}$ to $\frac{1}{2}$ inch is preferable. Railroad work of this character is usually constructed of $\frac{3}{8}$ -inch metal for ordinary depths.

The pivot pier of the bridge over the Little Bras d'Or River in Cape Breton was constructed of seven metal cylinders braced together. The center tube was 4 feet in diameter, while the six outside cylinders were 3 feet in diameter. (Fig. 58.) The center pivot, about which the span revolves, rests on the center tube, while the track is supported by the other tubes, but resting directly on rolled beams covered with $\frac{3}{8}$ -inch plate.

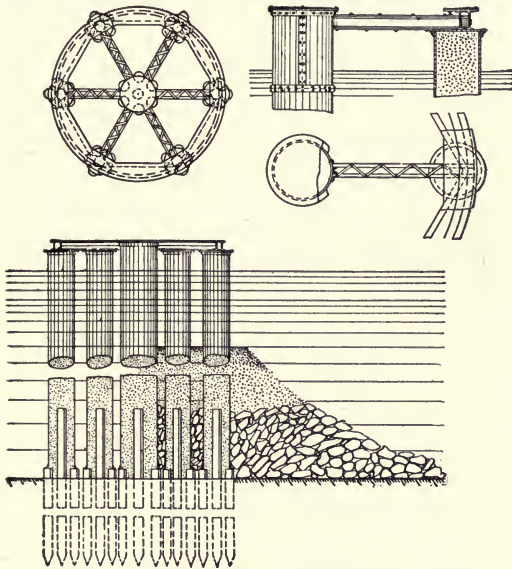


FIG. 58.—GROUP OF CYLINDERS FOR PIVOT PIERS.

The tubes rest on a clump of piles, cut off at the bed of the stream, with one pile extending up into the center of each tube about 6 feet, around which the concrete was deposited, thus preventing displacement. Concrete and stone were placed on the outside up to 15 feet, as a protection.

This work was described by Martin Murphy in *Trans. Am. Soc. C. E.*, Vol. 29, who also describes a pier for the Victoria Bridge, over Bear River, constructed with two tubes, resting on piles cut off at the bed of the stream, but having four piles inside each tube. (Fig. 59.) Around the outside are timber, concrete,

and broken stone as a protection. The saw used for cutting off the piles under the water was very much simpler than the one shown in Fig. 35, and is illustrated in Fig. 60.

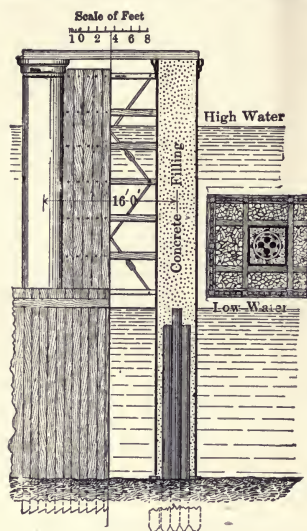


FIG. 59.—PIER OF TWO CYLINDERS, VICTORIA BRIDGE.

Cylinder piers on European work are often of very elaborate construction. The bridge on the Aa, at the crossing of the Russian Riga-Orel Railway, is supported on elegant cylinder piers, with molded caps, steel cutwaters, and are braced together with cylinders transversely. (Fig. 61.) This forms a very efficient construction, but so expensive to manufacture that it is usually replaced by bracing of struts and rods, as in Fig. 59, or by a metal diaphragm (Fig. 62) stiffened with angles.

Cylinders of water-tight construction and of large diameter may be used as coffer-dams where they are sunk into impervious strata, or by sealing them with concrete around the bottom where they are placed upon smooth rock bottom. In the construction of lighthouses such cylinders have been placed upon clean rock bottom, through from 12 feet to 18 feet of water, and concrete deposited around the circumference of the base outside and inside to make them water-tight, after which they were pumped out and the foundation laid.

To withstand the pressure of any considerable depth of water the thickness and strength should be calculated and the construction carefully designed. Unless the depth of water exceeds 10 feet, or the diameter of tube exceeds 6 feet, the minimum thickness, it is advisable to use, will be sufficient for strength.

This refers only to quiescent pressure, and any shock must be carefully considered and taken account of, by interior bracing if necessary.

The most thorough discussion of the strength of thin, hollow

metal cylinders is given in "Elasticität and Festigkeit," by C. Bach. This considers the cylinder to have sides of a greater thickness than is true with pier shells, and having one radius given, the radius to the other side of the plate is found from the formula, the stress being variable from the inside to the outside of the plate.

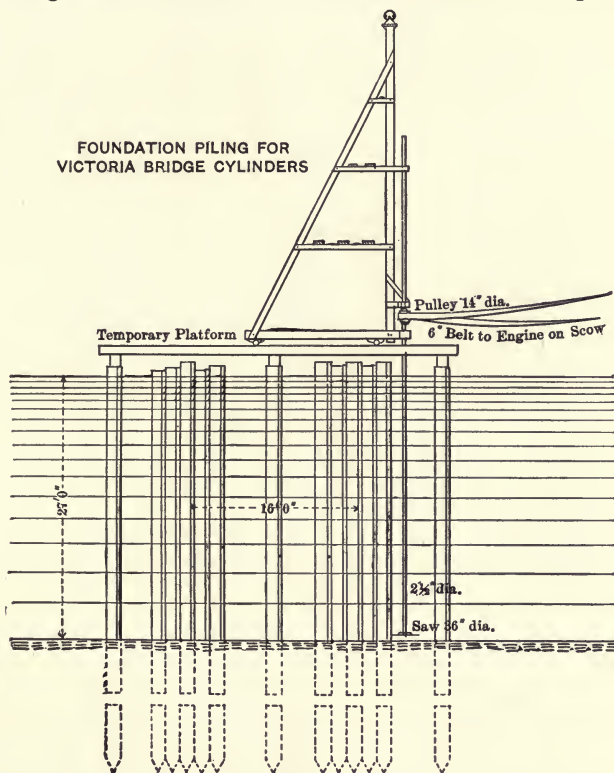


FIG. 60.—CIRCULAR SAW FOR CUTTING OFF PILES UNDER WATER.

For thin cylinders the stress may, without appreciable error, be assumed to be uniform over the cross-section of the plate, and the thickness t in inches be found from the formula

$$t = .001rh,$$

where r is the radius of the cylinder in feet, h is the depth of the water to the section in feet, and t in no case to be used less than $\frac{1}{4}$ inch in thickness.

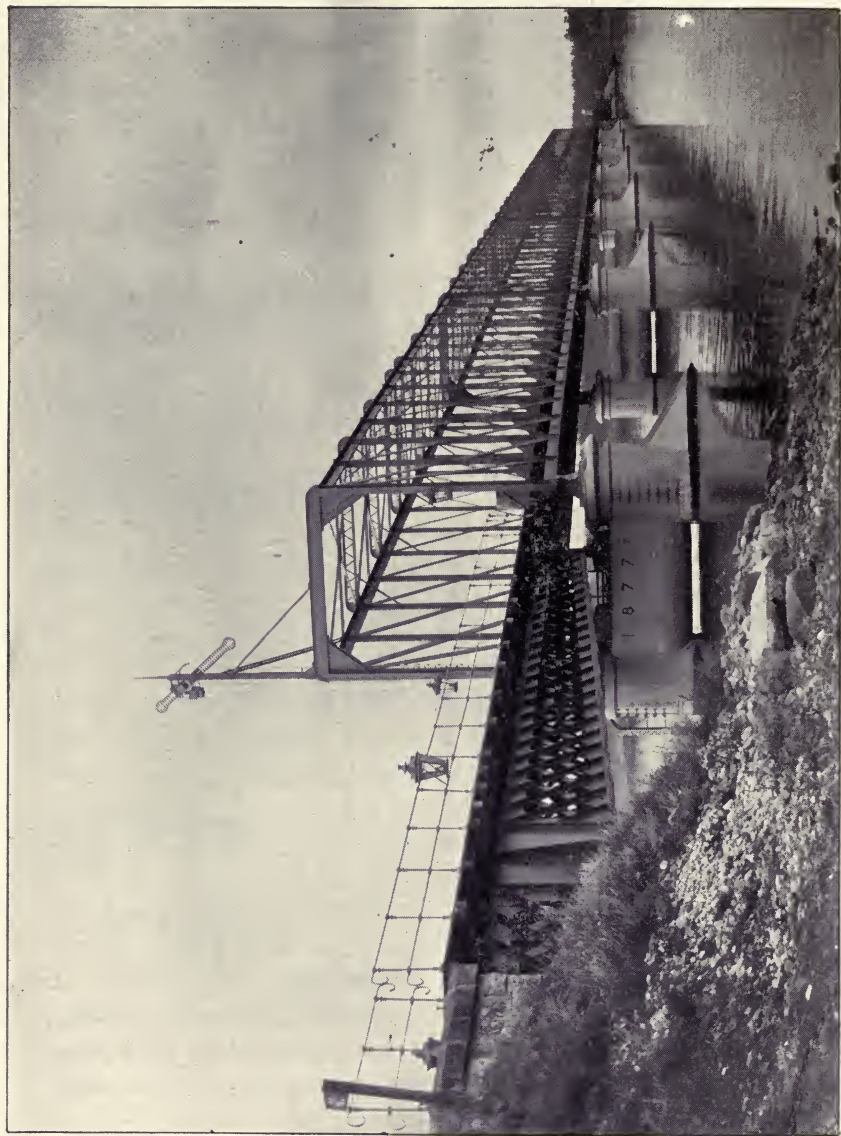


FIG. 61.—CYLINDER-PIER BRIDGE, RIGA-OREL R. R., RUSSIA.

This is on the assumption that the metal will stand 5,000 pounds per square inch in compression with safety. For large cylinders, or for rectangular shells, girders and stiffeners or ties and struts must be added to prevent distortion.

The foundations for the great Forth Bridge, which were con-



FIG. 62.—CYLINDER PIERS, WITH DIAPHRAGM.

structed under the direction of Sir John Fowler and Sir Benjamin Baker, required the use of various methods to reach solid bearing, as the enormous weight to be carried required the most substantial piers obtainable.

The use of coffer-dams of metal for the Inch-Garvie piers is described by *Engineering*: The site of the two north or shallow piers being wholly submerged at high water, and about half in the case of the northeast and three-fourths in the case of the northwest pier, submerged also at low water, the preliminary work was tidal, and between spring tides no work could be carried on at all at this place. When it is considered how exposed the position was there—the work having to be carried on upon

a narrow ledge of rock attacked by wind and waves from all sides—it will be understood that the progress could not be very rapid. The conditions of the contract here required that the rock should be excavated in steps, and that the rubble masonry comprising the foundation of the circular granite piers (Fig. 63) should be

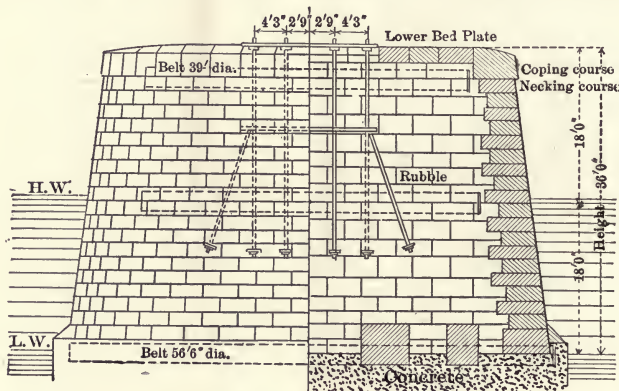


FIG. 63.—CIRCULAR GRANITE PIER AS FOUNDED BY COFFER-DAM, FORTH BRIDGE.

bound by an iron belt 60 feet in diameter and 3 feet deep; the highest portion of the rock upon which this belt rested to be 2 feet below low water; the belt, or at any rate a part of it, to be brought down to form a protection for the foundation rubble masonry upon the lower steps.

It was therefore decided to cut a chase 8 feet wide (3 feet to the inside and 5 feet to the outside of the 60-foot circle) out of the rock where it was higher than 2 feet below low water, to make the 60-foot belt of three thicknesses of $\frac{1}{2}$ -inch plate and to carry the center plate downward, after it had been cut, in such a manner as to fit as nearly as possible the natural contour of the rock. (Fig. 64, A.) A light staging was, therefore, erected above high water, the correct center of the pier placed upon it, and by means of a trammel-rod 30 feet in length, from the end of which a pointed sounding-rod was suspended, a correct reading was taken every 6 inches on the circumference of the 60-foot circle, after a diver had been around to clear out any loose stones lying in the line, or picking off any sharp points projecting. These

readings were plotted and the center plates cut to it. In the meantime work had been done upon the chase, and, when nearly cut down to the right level, the belt was put together on the staging exactly above the site of the pier. The plates, projecting

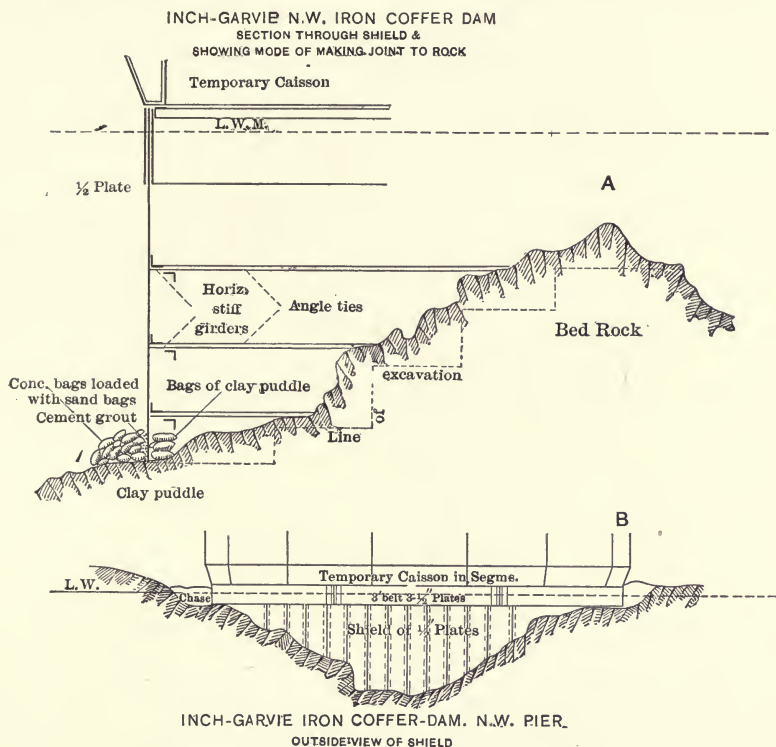


FIG. 64.—FORTH BRIDGE. METAL COFFER-DAM.

downward and forming the shield, were stiffened by I bars vertically over the butts, and where required to be carried down to a considerable depth, as in the case of the northwest pier, they were further stiffened by horizontal circular girders and stayed to the rock by bars of angle-iron. The whole belt was now riveted up, and when ready received two coats of red-lead paint, and was lowered down to position by means of hydraulic jacks. (Fig. 64, B.) The top edge of the 3-foot belt was then leveled all round, and corrected where necessary. A heavy angle-iron 6"

$\times 6'' \times \frac{7}{8}''$ ran round the inside of the 3-feet belt, and upon this was now set a single tier of temporary caisson, 10 feet in height, and consisting of fourteen segments of about 30 cwt. each in weight. This helped to keep the belt down to the rock, and a number of heavy blocks of stone were placed on the top of the caisson for the same purpose. A sluice door in the lower part was kept open to admit of the tide flowing in and out.

Steps were now taken to make good the joint between the 3-feet belt and the shield and the bed-rock. This was done in the following manner: A number of concrete bags, about $14'' \times 30''$, and $8''$ to $9''$ thick, were prepared and passed down to a diver, who laid them round the outside of the belt at a distance of about $4''$. A second row was laid next round the outside of the first row, and tolerably close up, the space between the two being made up by clay puddle well stamped down. Any split or hole or crevice in the rock was also filled with clay. Upon these two lower rows other bags were now laid crosswise; upon these, two rows lengthwise, and a fourth row crosswise on the top, which was laid close up to the belt. This was done in sections of about 15 feet to 16 feet length all along the shield, but round the outside of the treble belt only two bags deep were laid. On the inside also a single row of clay bags, backed by a row of concrete bags, and loaded with stones, was laid round the complete circle. Cement grout, without intermixture of sand, was now prepared and passed down to the diver—but only at slack tide, high water, or low water—who lifted off one or more of the top bags and poured the grout into the narrow space left, until it overflowed. He then replaced the bag and proceeded to the next division, until all was done. Forty-eight hours were allowed to elapse for the setting of the cement; the sluice valve was then closed and the caisson pumped out gradually. When leaks were discovered the diver descended to examine the outside, and where necessary, cut out some of the grouting and replaced it by new.

As it was not considered that this cement joint would be able to stand the full pressure of the tidal rise the coffer-dam was

worked as a half-tide one, it having to be pumped out every tide as soon as the water had fallen below the top edge of the temporary caisson. In addition to the hydrostatic water pressure, the caisson had to stand the heavy seas thrown against it, whether coming from east or west. Under these circumstances it was often considered advisable not to pump out the coffer-dam, but leave the sluices open and allow the tidal flow free access. Under such conditions it will be easy to see that, during a season of bad weather, much delay could not be avoided, and though the work of excavation had been commenced in the summer of 1883 it was not till the middle of April of the following year that the first rubble masonry could be laid in this pier. In working the excavation no blasting was done within $1\frac{1}{2}$ feet of the iron belt, but the rock was quarried up to within 6 inches and the rubble then built in at once. Any steps cut in the deeper portion were invariably at least twice as broad as they were deep. The deepest point to which the excavation had to be carried in this pier was 8 feet below low water.

The coffer-dam or caisson for the northwest pier, Inch-Garvie, was done in the same way precisely as described for the northeast, only that owing to the experience gained by the divers and other men engaged upon the work the progress was much more rapid.

In the northwest pier the depth of the shield was 15 feet below low water, and extended to nearly one-half of the circumference. There was, therefore, in addition to the vertical I bars which covered the butt joints of the shield-plates, three horizontal circular girders, carried at a distance of 4 feet 6 inches from each other; and from these a number of horizontal tie-bars with cross-bars at the ends were carried radially and level to the rock opposite and pinned to it, and afterward built into the solid rubble masonry. (Fig. 64, B.)

This mode of making the joint between the rock and the iron belt was simple and quite effective. Most of the leaks were due to natural crevices in the rock, running from the inside to the outside at a considerable depth. These were circumvented by building small clay dams round, and leading the water by a

chute to the pump. Leaks were also caused by the action of heavy waves running up to the temporary caisson at low water with great violence, and shaking the whole fabric.

The whole of the northeast pier was built in a half-tide caisson, as the work was not pressing; but in the case of the northwest pier, as soon as the rubble masonry inside had been brought up to a low-water level a second tier of temporary caisson was

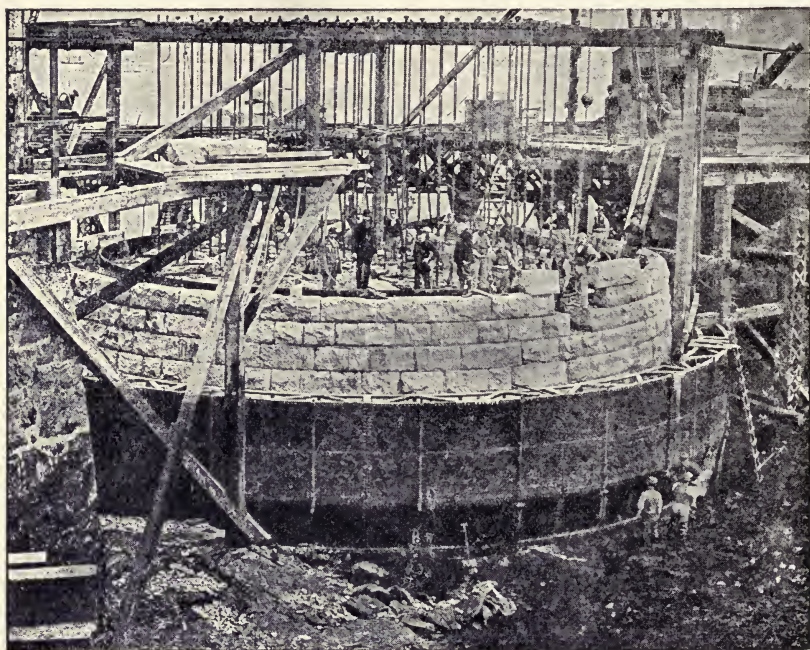


FIG. 65.—FORTH BRIDGE. CIRCULAR GRANITE PIER AND METAL COFFER-DAM.

added, and the work could then be carried on at all states of the tide. While tidal work was carried on in these two coffer-dams the amount of water which had to be pumped out every tide was 250,000 gallons in the one case and 340,000 in the other. The time occupied was 50 to 55 minutes, but work was, of course, commenced as soon as the higher parts were laid dry. For pumping out smaller quantities of water collected through leaks, pulsometers or small centrifugal pumps were used.

An exterior view of the work is shown in Fig. 65, and while the method was successful and worthy of much study, the expense would only be justifiable where the metal would be retained as part of the permanent foundation, which was the case on this work.

In many cases such a shell could be designed of the proper size for the footing course, and after use as a coffer-dam in obtaining the foundation it could be filled with concrete and serve as a base for the pier. Being made in sections vertically, portions projecting above low water could be removed and used on still other piers.

Metal sheet-piles were used on some harbor work at Cuxhaven Harbor, Germany. These were hollow metal sheet-piles of elongated, elliptical sections; and, after being driven, were filled with concrete.

Metal sheet-piling is being largely used in the United States, mainly of the Friestedt patent type. This is described in the *Engineering News* as follows:

"In many foundation works, particularly in quicksand and wet ground, the ordinary timber sheet-piling cannot be used to good advantage, and on works of this kind the use of steel sheeting is now being introduced. Fig. 66 represents a style of steel sheet-piling which has been used for the sheeting of foundations, mine-shafts, and also for coffer-dams, locks, etc.

"The cut clearly shows the construction of the sheeting and also two arrangements for corner construction. The piles consist of ordinary 15-inch, 33-pound rolled channels (with metal $\frac{3}{8}$ inch thick), each alternate channel having riveted to it two steel Z bars, forming grooves to receive the flanges of the adjacent channels. These bars are $4'' \times 3'' \times 3''$, $\frac{3}{8}$ inch thick. In this way the piles are so firmly interlocked that a line of sheeting will resist heavy pressure without the aid of shoring or bracing. In water the joints are soon calked by the accumulation of mud, sand, or dirt, but if the water should be very clear, a little sawdust, paper, pulp, manure, etc., may be thrown in near a leak, which will soon be sealed. In the Mississippi River coffer-dam

at St. Louis, noted below, the alternate piles had two angle-irons $\frac{5}{16}'' \times 2\frac{1}{2}'' \times 2''$ riveted on (as shown in the cut) to form calking grooves in which strips of wood might be fitted. These, however, have been found unnecessary and have been omitted on part of the work.

"This steel sheeting has been used for sinking mine-shafts through quicksand, and for the foundations of the Union Traction Building at Cincinnati and the new Railway Exchange Building at Chicago. The latter will be a seventeen-story 'sky-

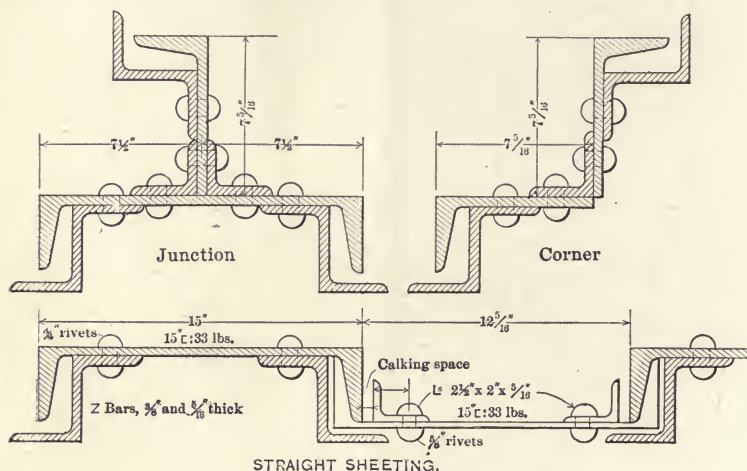


FIG. 66.—FRIESTEDT SHEET-PIILING.

scraper' office building, at Jackson Boulevard and Michigan Avenue, and the site will be excavated to a depth of nearly 30 feet. As there is a bed of quicksand underlying the site, it was decided to completely surround it with a steel sheeting, driven to a depth of 30 feet, which will resist the pressure from the outside and so prevent flow or caving which might injuriously affect adjacent buildings. This work is now in progress. An ordinary pile-driver with a 2,000-pound hammer is used, the head of the pile being fitted with an iron cap having a wooden cushion and a wooden striking-block. The sheeting has been used also to form the coffer-dam for the new power-house of the Union

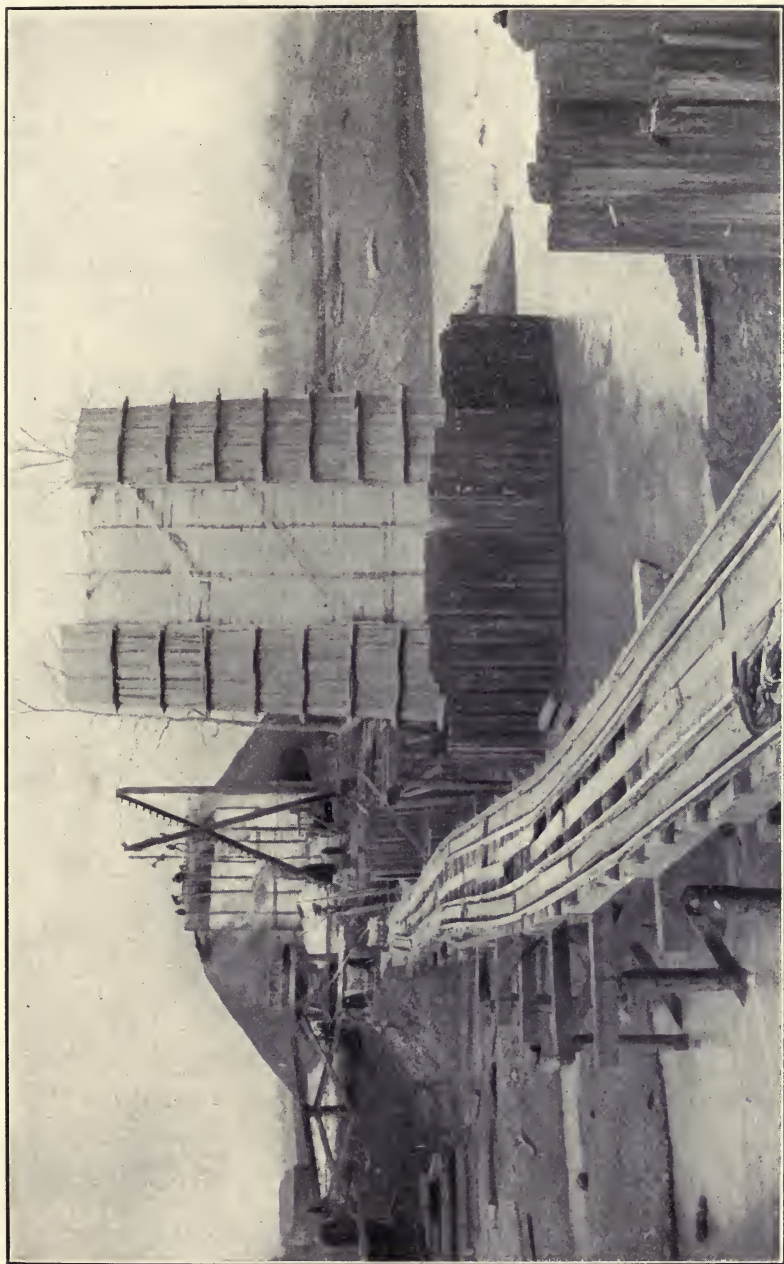


FIG. 67.—COFFER-DAM, C. B. & Q. RY., WITH FRIESTEDT PILING.

Electric Light & Power Co., at St. Louis (*Eng. News*, Nov. 6 and Dec. 18, 1902). The location is on the bank of the Mississippi River, and a coffer-dam, $40' \times 360'$, was built of the steel piles, 50 feet long, braced only by cross-walls dividing it into panels or sections, $40' \times 60'$.

"The piles are rolled of any length up to 60 feet, and can be spliced for longer lengths. In many cases it can be pulled up and used over again, and discarded material will have a high scrap value."

The cross-section of piling is shown in Fig. 66, while Fig. 67 shows some work on the C., B. & Q. Ry., on which it has been used. Whatever the class and form of material it may be decided to use in securing a foundation by the coffer-dam method, the temporary construction should be so related to the permanent foundation that as much as possible of the material used and labor employed will be of service in the finished structure.

CHAPTER VIII.

CYLINDERS AND CAISSONS.

THE preceding chapter discussed to some extent the use of tubes filled with concrete for cylinder piers, and the details of their manufacture and placing will now be more fully treated.

It is customary in many bridge shops to make the vertical distance between rivet lines of tubes exactly 5 feet, thus making it possible to carry in stock plates for the manufacture of highway piers, these plates being $62\frac{1}{2}$ inches in width and of varying lengths equal to the circumference of various-sized tubes, plus the $2\frac{1}{2}$ inches for lap, as is also used for vertical lap.

These plates are then laid out; that is, the location of the rivet holes marked by standard wooden templates; after the plates are punched they are rolled in boiler bending rolls, and assembled in as great lengths of tubes as can be handled on the cars for shipment, or hauled from the railroad to the bridge site, or, in the case of large tubes, in as long pieces as can be erected. This is very often only one section in length with the vertical seam riveted.

The sections of tubes with lap-joints are alternately large and small, the diameter differing by a little over twice the thickness of the plates, so as to telescope (Fig. 68); the small section in every case having the outside diameter equal to that specified for the cylinders of the piers

The tubes also have holes to which the bracing between them is attached, which usually consists of struts and sway rods.

Railroad piers of this type are generally manufactured from material which is rolled to conform to a particular specification,

TABLE I.—TUBE WEIGHTS AND QUANTITIES.

Diam- eter of Tube.		Thickness and Order Length.				All Quantities Below for Two Tubes or One Pair.						
		1/4	5/16	3/8	1/2	Weight of one 5-ft. 0-in. Section.				Cu. Yds. 1 Foot Vertical.	Weight 1/4-in. Cap.	Weight 3/8-in. Cap.
						1/4	5/16	3/8	1/2			
Ins.		Ft. Ins.	Ft. Ins.	Ft. Ins.	Ft. Ins.							
15	A	4 1	4 0 1/2			448	560			.090	30	45
	B	4 2 1/2	4 2 1/2									
18	A	4 10 1/2	4 10			532	664			.130	40	60
	B	5 0	5 0									
21	A	5 8	5 7 1/2			616	770			.178	55	80
	B	5 9 1/2	5 9 1/2									
24	A	6 5	6 5			700	874			.232	70	105
	B	6 7	6 7									
27	A	7 2 1/2	7 2 1/2			780	980			.296	85	130
	B	7 4	7 4 1/2									
30	A	8 0	8 0	7 11 1/2		864	1084	1302		.364	115	165
	B	8 1 1/2	8 2	8 2								
33	A	8 9 1/2	8 9 1/2	8 9		948	1190	1428		.440	135	195
	B	8 11	8 11 1/2	8 11 1/2								
36	A	9 7	9 6 1/2	9 6 1/2	9 6	1032	1290	1554	2080	.524	155	220
	B	9 8 1/2	9 8 1/2	9 9	9 9 1/2							
39	A	10 4 1/2	10 4	10 4	10 3 1/2	1116	1394	1680	2242	.614	180	265
	B	10 6	10 6	10 6 1/2	10 6 1/2							
42	A	11 2	11 1 1/2	11 1 1/2	11 1	1200	1500	1806	2408	.712	210	310
	B	11 3 1/2	11 3 1/2	11 4	11 4							
45	A	11 11	11 11	11 11	11 10 1/2	1284	1604	1926	2576	.820	240	355
	B	12 1	12 1	12 1	12 1 1/2							
48	A	12 8 1/2	12 8 1/2	12 8	12 8	1364	1710	2052	2744	.930	270	400
	B	12 10	12 10 1/2	12 10 1/2	12 11							
54	A	14 3 1/2	14 3 1/2	14 3	14 2 1/2	1532	1920	2304	3080	1.178	345	510
	B	14 5	14 5 1/2	14 5 1/2	14 6							
60	A	15 10 1/2	15 10	15 10	15 9 1/2	1700	2124	2556	3408	1.454	425	630
	B	16 0	16 0	16 0 1/2	16 0 1/2							
66	A	17 5	17 5	17 5	17 4 1/2	1864	2336	2802	3746	1.758	510	760
	B	17 7	17 7	17 7	17 7 1/2							
72	A	19 0	19 0	18 11 1/2	18 11 1/2	2032	2546	3056	4080	2.094	605	900
	B	19 1 1/2	19 2	19 2	19 2 1/2							
78	A	20 7	20 6 1/2	20 6 1/2	20 6	2200	2750	3306	4416	2.458	710	1055
	B	20 8	20 8 1/2	20 9	20 9 1/2							
84	A	22 1 1/2	22 1 1/2	22 1 1/2	22 1	2364	2960	3554	4746	2.850	830	1235
	B	22 3	22 3 1/2	22 3 1/2	22 4							

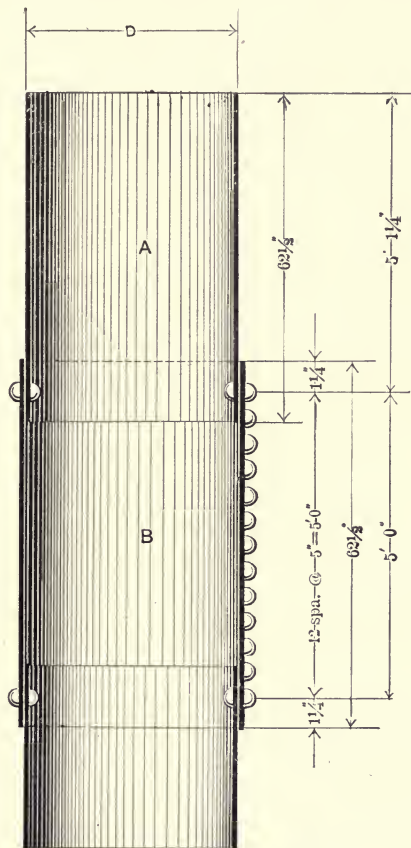
NOTE.—To get weight of *shells* for one pier divide height of pier by 5, multiply by weight of section, add in caps (which include cap-lugs).

Per cent. of rivets: For 15"×1", 3%; 48"×5/16", 2 1/2%; 84"×1/2", 2%. For other sizes use value for smaller size.

and the sheets can be ordered of such widths as will best suit each particular case. The tubes are made with butt joints and with much closer rivet spacing than is used for highway piers; that is,

in place of 4 to 5-inch spacing, they are spaced usually from 3 to 4 inches centers.

Frequently angle-rings are riveted around the tops of the



NOTE:- Make tubes of 62½ sheets, plus one narrower ring.

Horizontal spacing about 5" in:

Locate connections from horizontal gauge lines.

FIG. 68.—ORDER DIAGRAM CYLINDER PIERS.

tubes, and sometimes around the bottom to stiffen them, and in some cases are used to attach cap-plates to cover the tops, but it is better practice to carry a richer concrete above the top several inches and round it off around the edge to shed water.

The erection of the tubes to form piers is very often left to careless or incompetent foremen; but while the design of the metal work may be easily carried out to conform to some standard, each case of erection requires special treatment, and should be placed in careful and experienced hands.

Where the bottom is soft, the material should be removed to a harder stratum or to a considerable depth before the tubes are set in position, the placing of them being carried out by means of a gin-pole, a derrick, or a gallows-frame, as may be found necessary.

Should the hard stratum not be reached before the tubes are set, the excavation must be proceeded with by excavating inside by hand; or if this becomes impossible, by using machinery, such as a small orange-peel dredging-bucket which is small enough to go inside the tube. Very soft material may be removed by pumping it out with a centrifugal pump. Where the material is too hard for the tube to sink of its own weight as the excavation proceeds, then it must be weighted in some way. This has been done by adding a section or so to the tube, or by building a platform around the top and loading it with the excavated material. Where even then the cylinder sticks, a water-jet used around the outside may overcome the skin friction and assist the sinking.

The tubes need only be sunk to such a depth as will insure against scour, as shown in Fig. 69, unless by going a reasonable additional distance hard enough material may be reached, so that within the area of the cylinders of one pier sufficient bearing capacity will be had.

Otherwise piling in sufficient number must be driven inside each tube to carry the load, and these must be cut off near enough to low-water line to insure their keeping constantly wet to prevent rot. Enough space must be left between the piles, and between the piles and the metal shell, in which to tamp concrete.

Another plan very often used for supporting cylinder piers on soft ground is to drive piles to carry a grillage on which to set and fasten the tubes. The grillage may be drift-bolted to the piles, which have been cut off to a level, or the grillage may be fastened

to the bottom of the tube and the whole then lowered onto the piles; guide-piles around the sides being used to insure the proper placing, and to anchor the tubes in position. With such a foundation, riprap stone should always be used for protection from the current of the stream, and riprap should always be used wherever there is any possibility of scour.

Where the pier is placed in the current a crib should be placed around it, if protection is necessary, and the space between the

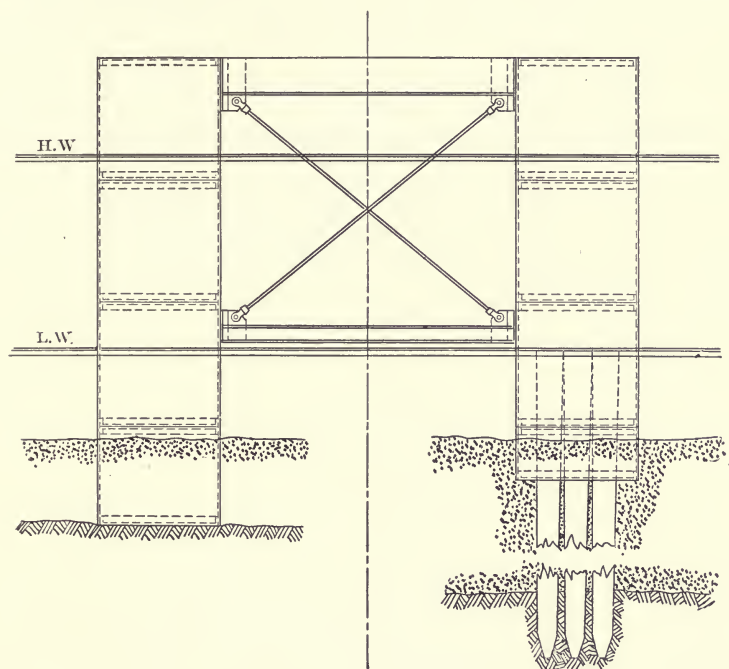


FIG. 69.—DESIGN FOR CYLINDER PIERS.

crib and the cylinders filled with riprap. (Fig. 70.) Enough riprap should also be placed outside of the crib to protect that from scour.

Where there is solid rock on which to place the cylinders, they should be anchored to it in some way. In shallow water this is usually done by placing a coffer-dam around the pier so that the rock can be leveled off whatever it is necessary, and anchor bolts

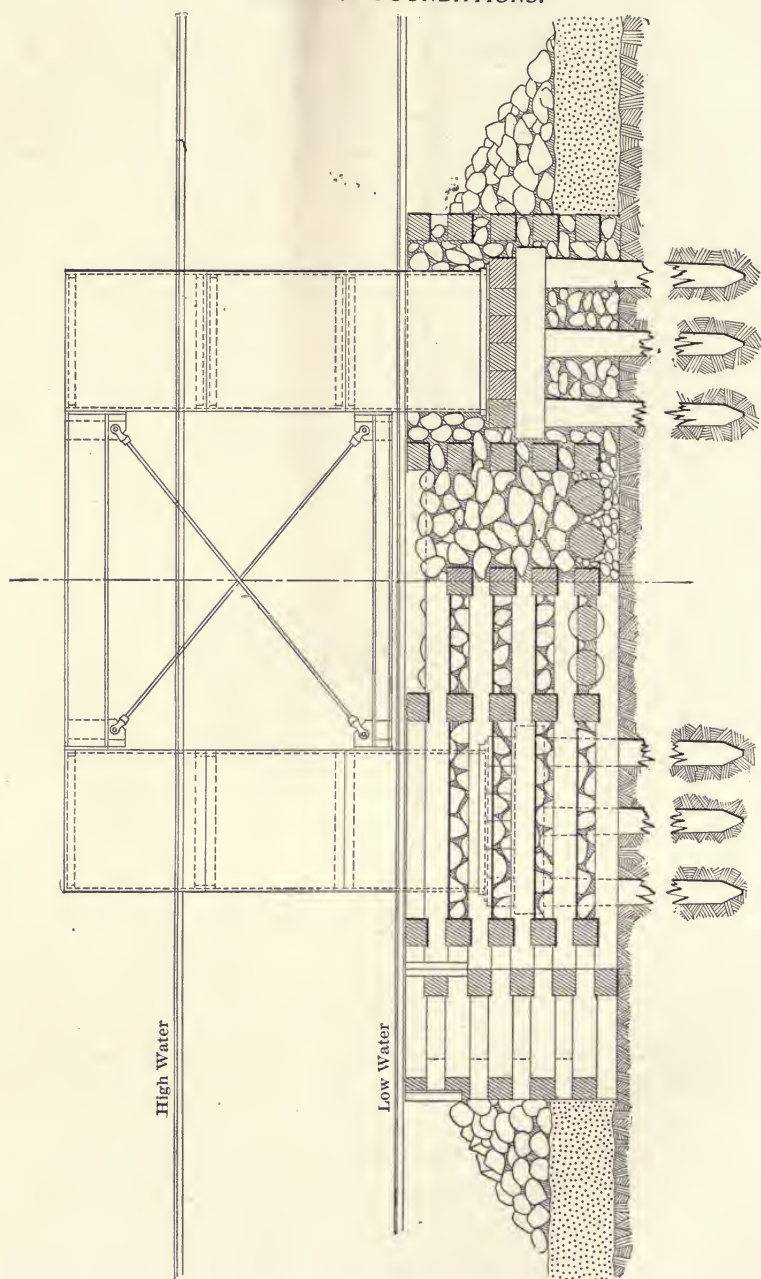


FIG. 70.—CYLINDER PIER WITH CRIB.

used to anchor the cylinders in place, the anchor bolts passing through lugs on the bottom, or through the legs of an angle-rim. If the rock is at all uneven and it is impossible to lay it bare to do the leveling off, then the bottom of the tube should be cut to fit the irregularities as they are learned from soundings. Anchors may be placed by drilling into the rock and setting old railroad rails into the drill holes, allowing the rails to project up several feet into the cylinders. Then when the cylinders are placed, concrete may be deposited in them under water by some of the approved methods.

It is very often advisable to put a crib around a pier on solid rock in order to protect it against the force of the current, drift-wood, and floating ice.

Considerable mention has already been made of piers being sunk by dredging out inside either tubes, such as have just been treated of, or larger metal piers, such as were used for the Hawkesbury Bridge. In the United States, owing to the fact that timber is so plenty and cheap, it has been the practice to use piers similar to those of the Hawkesbury Bridge with the caisson and crib of timber instead of metal. An example of this type of pier is shown in Fig. 71, which was designed by the author for a draw pier for the Northern Pacific Railway Company. The outside diameter of the octagonal crib being 38' 4'', and the total height of the pier being 84' above the cutting edge. The caisson proper is built up solid of timber with one cross-cutting edge, which was deemed advisable on account of the large diameter of the pier. The height of the caisson to the roof is 8' 6'', while the roof consists of three layers of 12×12 timbers, each of which is thoroughly calked and pitched. The sloping sides of the caisson walls are stepped to avoid the bulging pressure during sinking, instead of straight as has been customary on work of this character. This is a detail of construction which has been developed by J. A. L. Waddell, M. Am. Soc. C. E., on large work of this character which he has completed and which will be mentioned more fully in this chapter.

Above the caisson a timber crib is carried up to low water, and

the outside of the caisson and of this crib are thoroughly calked, covered with tarred ship-felt and planked, the ship-felt and

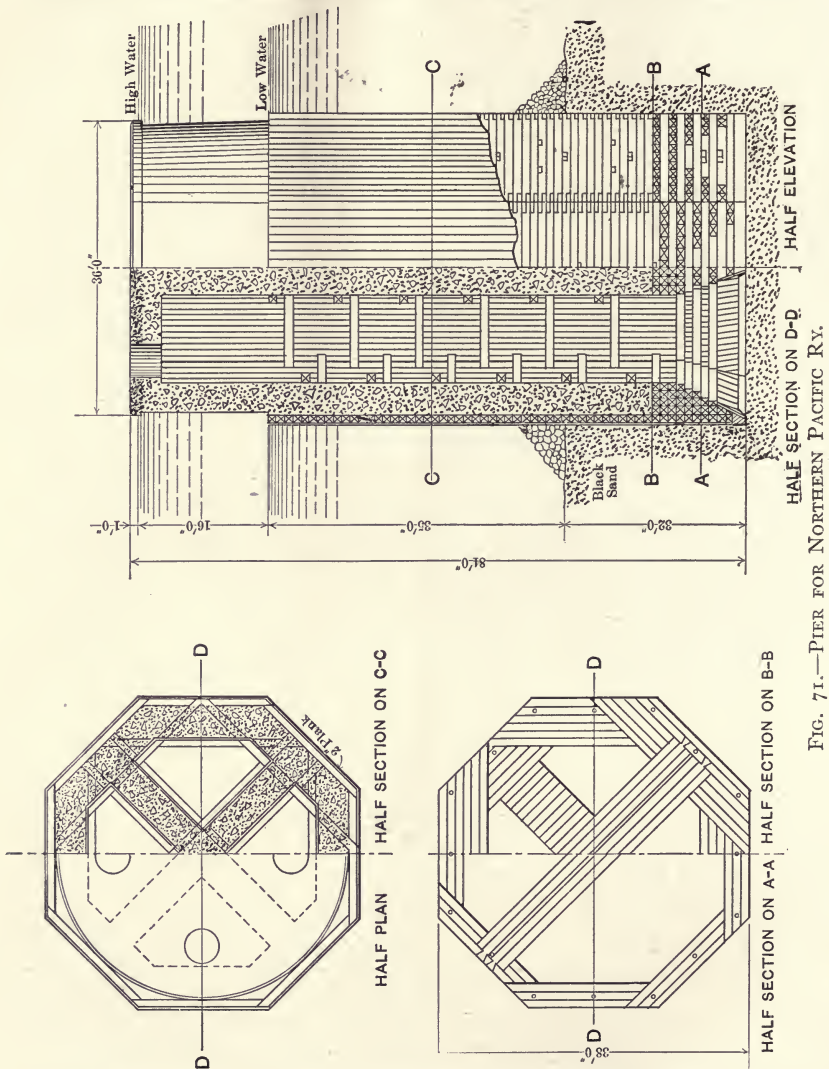


FIG. 71.—PIER FOR NORTHERN PACIFIC RY.

plank serving as additional protection for water-tightness and as protection against the teredo, while the planking also very materially strengthens the crib. Above the top of the crib a temporary

crib coffer-dam is used to exclude the water while laying the concrete, although it is possible, on account of the 16 feet range of tide, to lay most of it in the dry with ordinary concrete forms.

Instead of dredging out the material with sand-pumps of the ordinary type or with dredging-buckets, the Hendy hydraulic elevators No. 5 are used, Fig. 72, with a water pressure of 125 pounds. An elevator of this sort can be shifted around and the material dredged out of different parts of the caisson, so as to cause the pier to sink evenly. But, should it be impossible, for any reason, to keep the pier plumb, this can be corrected by the permanent water-jets, as indicated in the sections of the pier. Some of these jets come out at the cutting edge and some of them along the sides of the pier. It has been found possible, however, to accomplish better results by using separate jets at such points on the outside as may be necessary. Should the jets fail to correct the trouble, then it is necessary to resort to the use of dredging on the outside, although this is not very often resorted to. With a pier of this sort the caisson is constructed on shore up to such a height as will bring the top of the timber above the water after it is launched and floated. (Fig. 73.) It should then be carried up rapidly enough, so that as the concrete is filled in it will be kept above the water until the pier is landed on the bottom as it is sunk. Only sufficient concrete of course can be put into the pier to keep it sinking properly, and with the crib calked and water-tight, it can be pumped out and the concrete laid in the dry. The concrete specified for this work was one part of Portland cement, three parts of sand, and five parts of screened and washed gravel. A facing of mortar 1 to 3, with a thickness of $1\frac{1}{2}$ inches, is specified for the facing, and the entire coping of 1 to 2 mortar reinforced with wire-netting and railroad rails, so as to carry machinery at any point.

Upon the pier reaching the proper depth and the wells being cleared of all material, they are filled to about 40' above the base with concrete, which is first put into the caisson through the water until the caisson is sealed. Then the wells are pumped out and the balance of the filling put in in the dry.

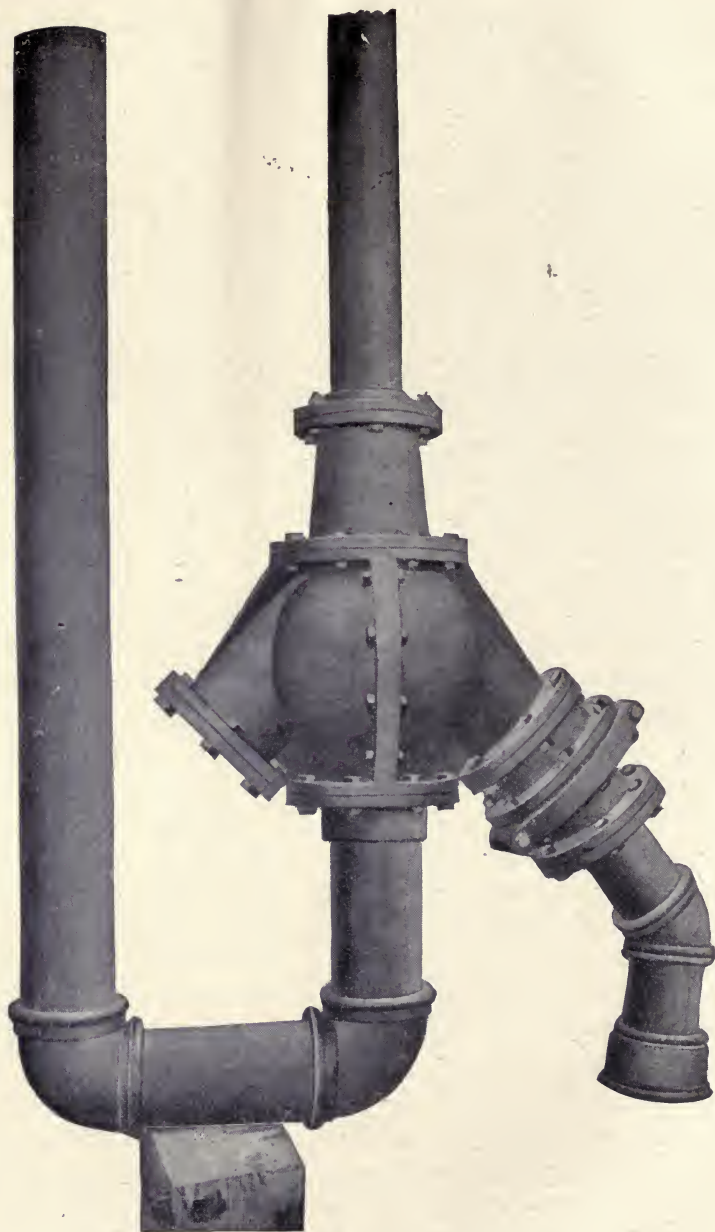


FIG. 72.—HENDY HYDRAULIC ELEVATOR.

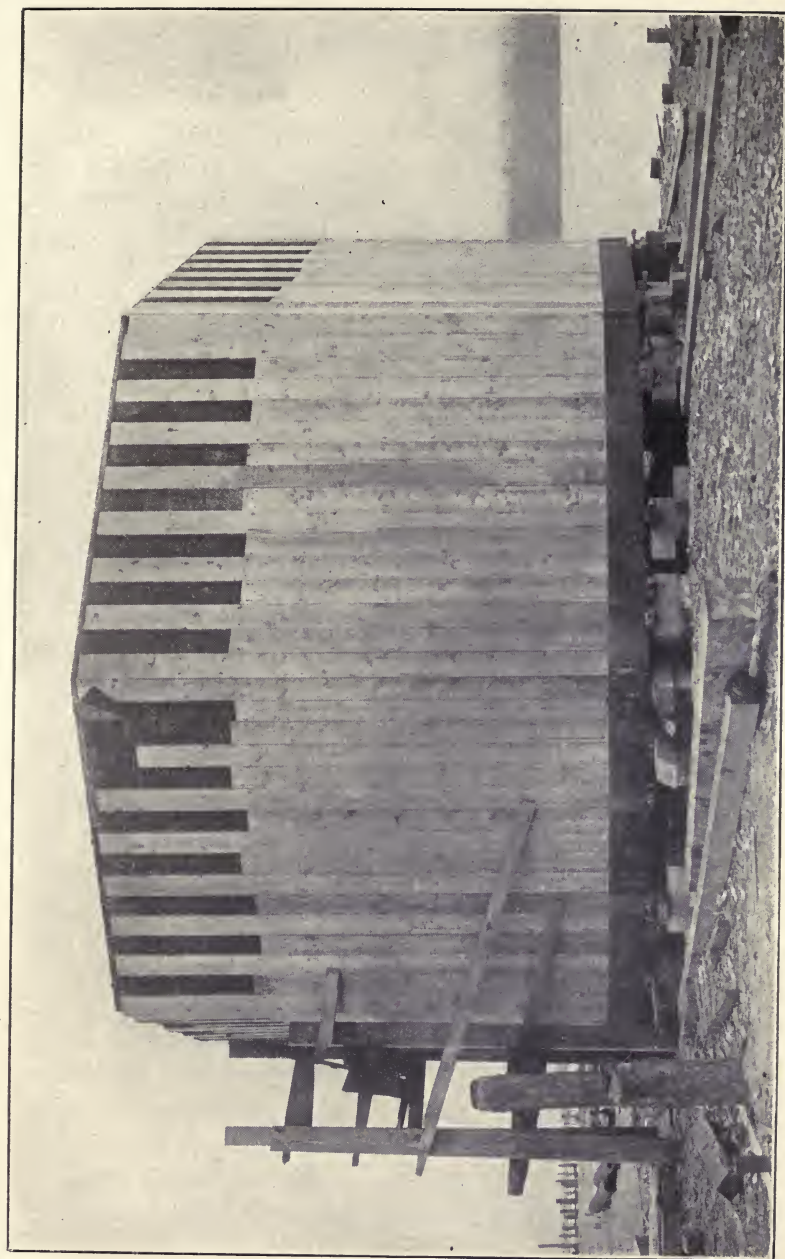


FIG. 73.—CAISSON NORTHERN PACIFIC PIER.

Piers of very similar design were used for the bridge over the Fraser River, at New Westminster, B. C., under J. A. L. Waddell, consulting engineer. There were five piers of this type, some of them having a total height exceeding 125'. The endeavor was made in the design of the caissons and cribs to deposit as nearly all of the concrete as possible in the dry. All the timber work was thoroughly calked and, on account of the calking on the roof timbers being on the opposite side from what was necessary for resisting the pressure, pitch was used on all the seams, consisting of crude resin mixed with a sufficient quantity of tallow so that it would be stiff and yet not too brittle. This was mixed in a kettle and the ordinary pitch vessel used for pouring it into the seams. In one of the first piers constructed only the seams of the 2" sheathing were calked, and there was considerable leakage; so that all of the seams in the other cribs were calked, including the 12 × 12 timbers. The experience had on this work points to the advisability of using two threads of oakum in all this calking. Where the jet-pipes pass through the roof of the caisson it was found necessary to seal the openings with pitch or a rich grout, and the entire surface of the rock was covered with an 8" layer of rich concrete, it being placed while the deck was above water-level. On account of the trouble had from leakage around the pipes, the later piers had these carried down through the well holes. In building up the cribs the timbers were lapped at the corners and well drifted, and were found to be stronger than the ordinary method of dapping. The solid timber portion of the work was drift-bolted at every crossing, thus tying the corners permanently together. When a small amount of penetration had been gotten on piers Nos. 4 and 5, the concrete work about the wells was carried above the surface of the water, and all timber work about the wells except the sheathing omitted. With pier No. 3, however, a great deal of trouble was found in holding the crib plumb, and it was never possible to carry the concrete to the level of the water-surface. Consequently solid well timbers were carried up to the very top of the crib. In sealing the wells the amount was regulated by the height of the crib. This was decided upon at 75 feet for pier

No. 5, and 50 feet of sealing was used; while for piers Nos. 3 and 4, lifts of 55 and 65 feet were used respectively, this sealing being allowed to set a week in each case. For any work of this character the caisson timbers should be very thoroughly fastened together and ordinary ship-building methods employed to the extent of using a great many through-bolts riveted down on clinch-rings. With a circular or octagonal crib the cutting edge should be spliced together at all points sufficiently strong to make a tension-ring about the bottom to take up the bulging pressure.

Pneumatic caissons cannot be considered as coming strictly under the head of ordinary foundations, although there are many cases where they could be employed much more cheaply than the methods which are finally adopted, besides assuring a first-class piece of work, where with some other method there may be more or less guesswork about the result obtained. In the construction of an ordinary highway bridge at Chillicothe, Ohio, in 1898, after designs by the author, it was decided by the engineer, A. W. Jones, to employ pneumatic caissons in founding the piers, as it was possible to let the contract for slightly less than \$13 per cubic yard of caisson and crib, or not very much in excess of what any other type of pier would have cost. One of these piers is shown in detail in Fig. 74, carried to bed-rock 30 feet below low water. The caisson of this pier was 7 feet 8 inches in width, 27 feet 8 inches long and 6 feet 2 inches high. The out-to-out dimensions were 12 feet 4 inches in width by 32 feet 4 inches in length. The sides and roof of the caisson were 2 feet in thickness, built up of 12×12 timbers, each course being fastened to the one below with drift-bolts 30 inches long and spaced 4 or 5 feet centers. The two courses on the sides and ends were firmly fastened together by two rows of $\frac{7}{8}$ -inch bolts extending through both courses. The two bottom layers of timber were beveled off on the inside to form a 4-inch "cutting edge," this being protected by a 2×6 plank firmly spiked on, which was very satisfactory, as all the timber was of oak. The working chamber was divided into three equal "pockets" by three tie-beams dovetailed into the sides of the caisson. The roof was

pierced by five holes; 4-inch holes near each end for the "blow-out" pipes and an 18- and a 20-inch hole in the center "pocket" for supply-pipe and air-shaft. Near the air-shaft was another

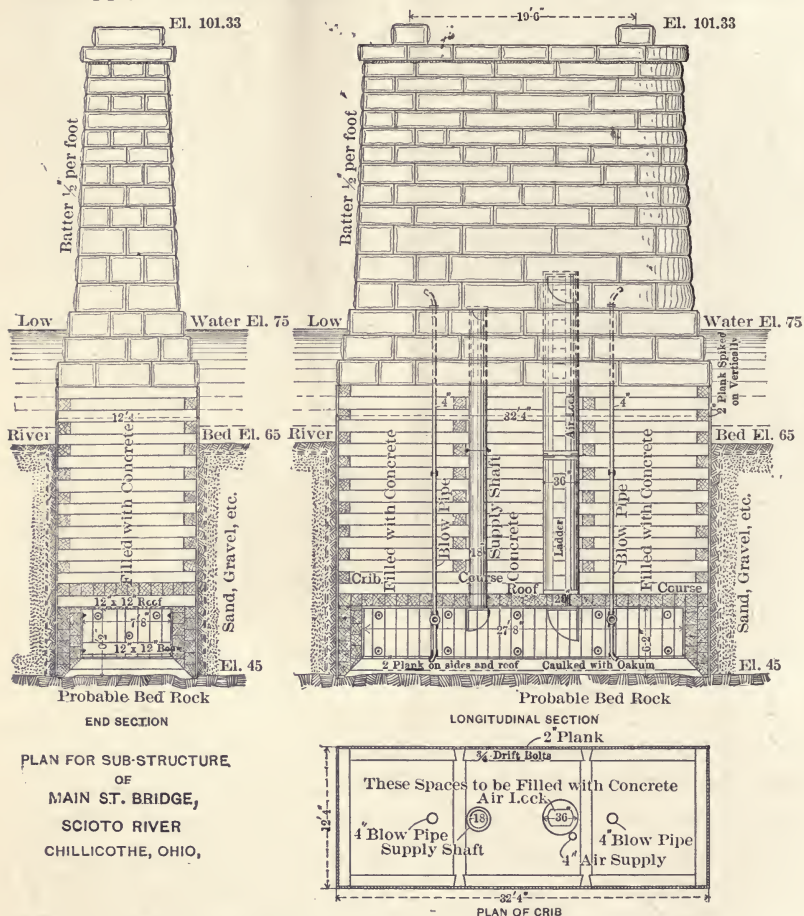


FIG. 74.—PNEUMATIC CAISSON, CHILLICOTHE, OHIO.

4-inch hole for the air-supply pipe. The air-shaft itself was formed of $\frac{1}{4}$ -inch boiler-iron thoroughly riveted and calked and fastened together with inside flanges bolted together with $\frac{3}{4}$ -inch bolts. Supply-pipe was formed of $\frac{3}{16}$ -inch plates, and was similar in construction to the air-shaft, except that the flanges were on the

outside. The inside of the caisson from the "cutting edge" to the roof and all over the roof was calked carefully in each joint

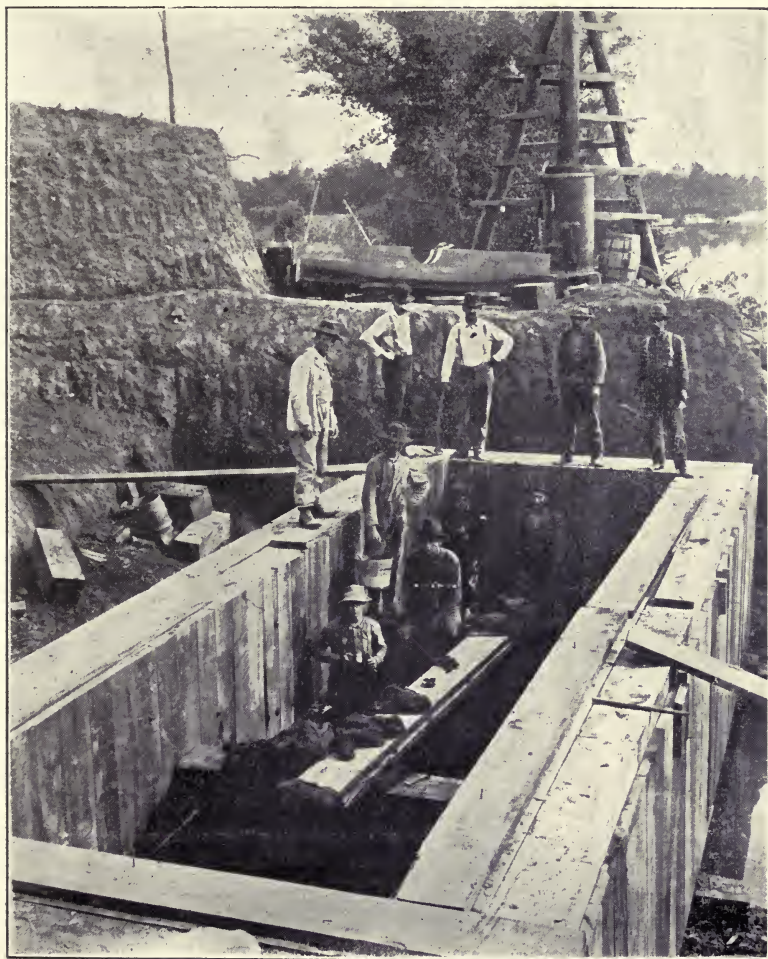


FIG. 75.—CALKING CAISSON No. 2, CHILLICOTHE, OHIO.

between the timbers and around the bolt-heads. Then the sheathing was put on and this was thoroughly calked (Fig. 75). The caissons were built on level ways on the river bank (Fig. 76), and when ready for launching these ways were inclined by raising

the inner end, and the launching was accomplished by starting the caissons with tackle until they slid off into the water.



FIG. 76.—CAISSON No. 1 ON WAYS, CHILLICOTHE, OHIO.

On top of the caissons cribs were built of 12×12 timbers with cross-ties in every other course and 30-inch drift-bolts

every 4 or 5 feet. This was sheathed continuously from "cutting edge" to the top. After grouting the deck or roof of the caisson several feet of concrete was put in until the pier became heavy enough to land on the bottom. The concrete was composed of 1 part of Portland cement, $2\frac{1}{2}$ parts of clean sand, and 5 parts of gravel, and the contractor was allowed to place large, clean bowlders in each layer.

The plant for sinking the piers consisted of a double compressor $14 \times 16 \times 18$, run by a 120-horse-power locomotive boiler. The air-receiver was 56 inches in diameter and 15 feet long, while the lighting plant consisted of a 115-volt dynamo of 9 amperes capacity run by a vertical engine. In addition to this there were all the necessary pumps, hoisting-engines, derricks, and the like for carrying on the work. A 3-inch pipe was run from the receiver, along the river bed, to each pier and connected to the supply-pipe by a flexible joint. The working chamber was lighted with three 16-c.p. electric lights, and one was placed in each section of the air-shaft. Work was begun as soon as the air was turned on, by working two shifts of ten hours each. The "sand-hogs," or crews of men employed on the work, consisted of one foreman, two blowpipe feeders, four shovelers, an inside lock tender and an outside lock tender. The material was removed by means of ordinary sand-pumps and was discharged through "goosenecks" of cast iron outside the pier, these "goosenecks" being heavy enough to stand the wear for a considerable length of time. All rock too large to send out through the pipes was taken out through the air-shaft in sacks. In sinking the caisson all material was cleaned out level with the "cutting edge," then it was "ditched" by shoveling several inches of material from under the "cutting edge" and giving a "blow" by opening one of the valves and letting out a greater part of the air. Relieved of the lift of the compressed air, and with the weight of the concrete on top, the caisson settled down from 6 to 10 inches until the "cutting edge" was again on solid gravel. While the air was escaping, the working chamber filled with water and would frequently be two-thirds full when done "blowing." No

particular trouble was had in sinking the piers except for caisson No. 1, where, at a depth of about 20 feet below the bed of the river, a layer of bowlders was encountered and several coping-stones from one of the old piers of the old bridge, which was being replaced. The bed-rock under this pier was reached at 43 feet below low water and consisted of black shale and lime-



FIG. 77.—SETTING FOOTING COURSES No. 2.

stone in alternating layers. After the caisson was landed the concreting of the air-chamber was begun.

To concrete the air-chamber a 2-inch pipe connection was made from the air-pipe to a point just below the top flange of the supply-pipe. This pipe had a valve, No. 1, near the air-pipe, and another valve, No. 2, on a T between valve No. 1 and the supply-shaft. A top door opening down was then put on the supply-shaft and the fastening taken off the lower door. A batch of concrete was shoveled into the supply-pipe, the upper door pulled up, valve No. 2 closed and No. 1 opened. This put compressed air in the supply-shaft, and allowed the lower

door to equalize open and the concrete fell into the working chamber, where it was rammed under the bevel edges and over the bottom by the "sand-hogs." The lower door was then shut, valve No. 1 closed, and No. 2 opened, allowing the compressed air in the supply-shaft to equalize out and the top door to open. The process was repeated until the chamber was filled with concrete and barely a narrow passageway for one man left between the lower doors of the supply-shaft and air-shaft. The lower doors were then shut, and the last man equalized out. A regular air pressure was then kept on for twelve or eighteen hours, until the concrete set. The air was then turned off, and the shafts filled with concrete as quickly as possible. As soon as the air pressure was taken off, the lower doors dropped and the space left in the working chamber also filled with concrete, which was run in from the shafts. This finished the foundation from bed-rock to above low water. The setting of the footing courses is shown in Fig. 77.

The above account is taken practically verbatim from the report of A. W. Jones, the engineer on the work, to whom acknowledgment is made.

CHAPTER IX.

PUMPING AND DREDGING.*

THE degree of success which has been attained in the building of a coffer-dam will be evident when the pumping process is begun. After having been pumped out, if the leakage is so small as to require only a small amount of pumping to keep it free from water, it may reasonably be considered a success.

The pumping should not exceed what can be done by a steam-siphon, a small pulsometer, or by running a centrifugal pump intermittently. Should leaks develop which cannot readily be contended with, then repairs must be made.

The use of pumps for this class of work on ancient bridges is described by Cresy. The bascule, used by Perronet at the bridge of Orleans (Fig. 78), is one of the most primitive forms. It consists of a seesaw apparatus, at each end of which ten men were placed, and 150 motions were given it in each quarter of an hour. Four cubic feet of water were raised 3 feet each time, or about 300 gallons per minute. Various other kinds of pumps were used at this bridge, among them the chapelet, which is similar to a modern chain-pump, worked by hand. Then the same device was employed, but geared to be operated by horses on a platform. A chapelet operated by a water-wheel was also

* Attention is called to the numerous references in other chapters of the pumping plants actually employed on coffer-dams, and especially to the plant used at Topeka, page 92.

Great care should always be given to the selection of a pumping plant of the proper type and proper size, as the statements regarding capacity are often misleading. The outfit should be, if needed, one able to take care of the dredging, if the material is such that it can be pumped.

used. (Figs. 79 and 80.) The large wheel had 124 cogs, while the pinion had 15, which caused the raising of over sixty-six buckets on the chain for each turn of the large wheel. At 180

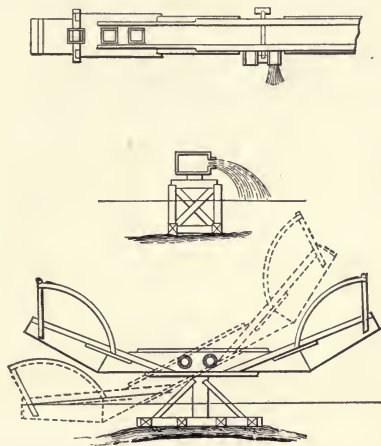


FIG. 78.—OLD BASCULE PUMP.

turns of the wheel per hour, with each bucket lifting 290 cubic inches of water, the capacity was about 250 gallons per minute.

A great bucket-wheel was employed by the same engineer at

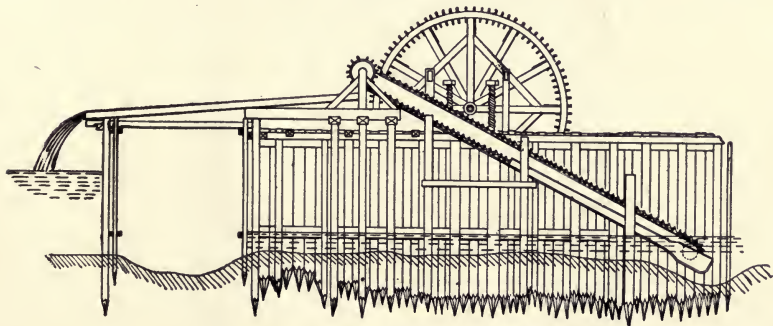


FIG. 79.—OLD CHAPELET, SIDE ELEVATION.

the Neuilly Bridge, 16 feet 6 inches in diameter, 4 feet 6 inches wide, with sixteen buckets.

The pumps used at the present time on very small work are usually square wooden-box lift-pumps, such as are used on large

river barges, and are worked by one or more men lifting on a plunger. These are often replaced by a similar pump of metal (Figs. 81 and 82) with a tube of galvanized metal, and often

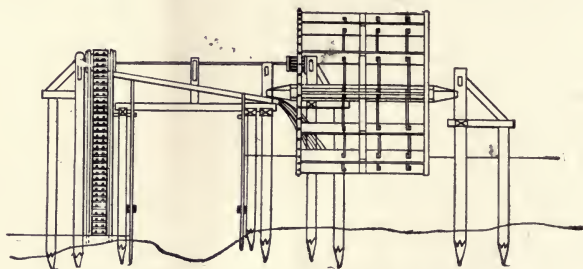


FIG. 80.—OLD CHAPELET, END ELEVATION.

spiral-riveted. The one shown in Fig. 81 has the top and bottom soldered to the tube, while the one in Fig. 82 has screw joints. The cost of a 4-inch pump 8 feet long with fixed top and bottom



FIG. 81.—HAND-PUMP,
SOLDERED JOINTS.



FIG. 82.—HAND-PUMP,
SCREW JOINTS.

would be about \$6, while the screw joints would about double the cost.

Such pumps are, however, little used, as the labor becomes

excessive where there is any quantity of water to deal with, and diaphragm-pumps (Fig. 83) are employed, which work on a rubber diaphragm in place of a piston and plunger, and throw a large amount of water, besides allowing the passage of sand and gravel without choking the pump. The $2\frac{1}{2}$ -inch suction has a capacity of 25 gallons per minute, and the 3-inch suction of 58 gallons per minute, the list price of the two sizes being \$20 and \$26, respectively; the maximum lift of the pump being 30 feet.

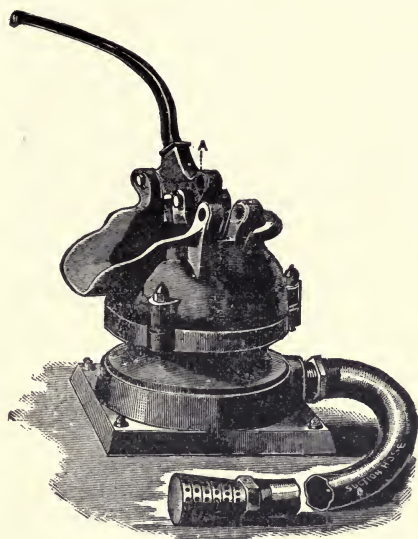


FIG. 83.—DIAPHRAGM-PUMP.

Where steam can be obtained steam-siphons are often used, the steam being introduced into the main pipe through a nozzle, thus causing a suction, which with a 3-inch discharge Van Duzen jet will deliver 7,200 gallons of water per hour, the height of the pump above water being 11 feet the point of discharge being 19 feet above the pump, making a total lift of 30 feet. This size will require an 18-horse-power boiler and a steam pressure of 50 pounds. The suction-pipe is 1 inch larger than the discharge, while the steam-pipe is $1\frac{1}{4}$ inches in diameter, with a jet opening of about $\frac{1}{16}$ inch.

The list price of a pump of this size (Fig. 84) is \$36, the piping being extra. The pump is constructed of gun-metal and will last indefinitely. The strainer should always be used and will cost about \$4 extra for the 4-inch pipe. The piping should have long bends in place of elbows where a turn is required.

This make of pump is manufactured from $\frac{1}{2}$ -inch discharge,

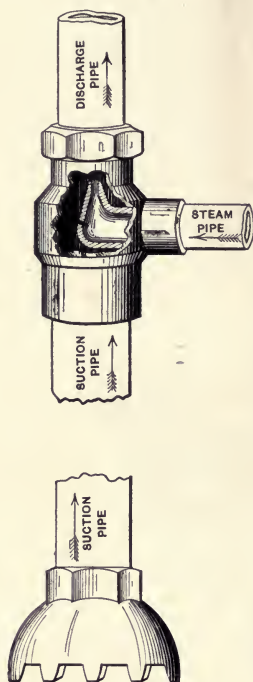


FIG. 84.—VAN DUZEN
JET-PUMP.

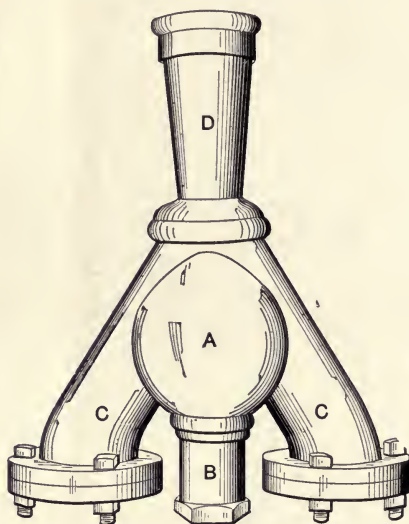


FIG. 85.—LANSDELL'S SIPHON-PUMP.

with a capacity of 200 gallons per hour, up to 5-inch discharge with a capacity of 12,000 gallons per hour. The smaller sizes are useful for priming centrifugal pumps and for a variety of uses around a contractor's plant.

The Lansdell siphon-pump (Fig. 85) has a double suction *CC*, to which rubber suction-pipes are attached. The steam-pipe is attached to *B*, and when the steam is turned on it is blown across *A* and through *D*, thus exhausting the air from the cham-

ber *A*. Water rises through *CC* by atmospheric pressure to fill the vacuum, and it is forced out through *D* by the steam, the velocity being proportional to the steam pressure. The steam supply should be as close to the pump as possible, to prevent condensation, and the turns in the pipe should be easy bends, as stated regarding the Van Duzen jet. When the height exceeds 14 feet to which the water is to be pumped, the suction-pipes must be long enough to allow the center of the pump to be placed 14 feet above the water. With a 3-inch discharge, a 1½-inch steam-pipe is required and a 12-horse-power boiler. With a 6-inch discharge a 2½-inch steam-pipe is required and a 50-horse-power boiler.

The rated capacity of the 3-inch is 450 gallons per minute, of the 6-inch 1,800 gallons. But this would likely not be realized in practice.

The vacuum-pump which has reached the most general adoption is the pulsometer, and is in many ways better adapted to light service than a centrifugal pump of small size. There are no bearings to keep up, no belts to keep tight, and no trouble in preparing a foundation, as the pump is suspended by the hook shown in Fig. 86. The pump is operated by admitting the steam through the pipe at the extreme top (Fig. 87), the pump having been previously primed by filling the middle chamber with water. The air-valves are closed and the steam passes into the right-hand chamber *A*, clearing it of water by forcing it into the discharge-chamber shown in dotted line. The steam then condenses at once and the ball *C* changes its seat, closing the right-hand and opening the left-hand chamber to the steam. The vacuum, formed by the steam condensing in the right-hand chamber *A*, allows it to fill with water by atmospheric pressure through the suction pipe at the extreme bottom and through the chamber *D*, it being retained by the valves *E*, *E*. The steam then enters the left-hand chamber *A* and the operation is repeated. The chamber *J* is a vacuum-chamber.

In starting the pump the steam is turned on for three or four seconds, then shut off for four or five seconds, alternating these

movements until the pump is started. The steam is then turned on about half or three-quarters of a revolution, the two side air-valves opened about half a turn, and then the middle air-valve opened slowly until a regular stroke is obtained.

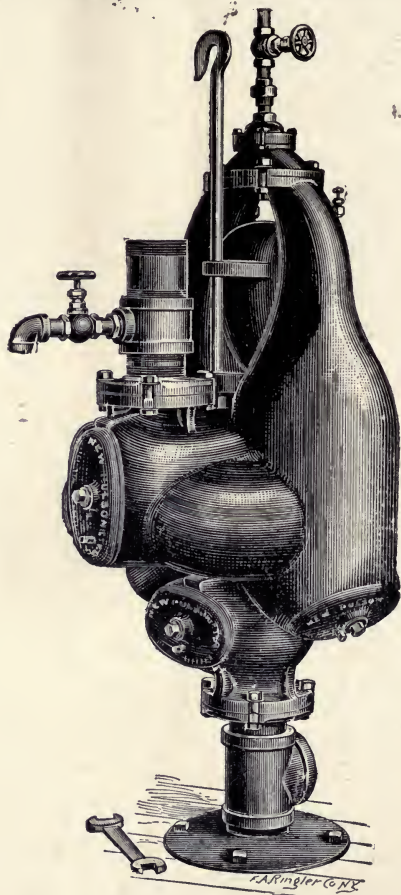


FIG. 86.—PULSOMETER STEAM-PUMP.

The capacity of the 3-inch discharge, with a $\frac{1}{2}$ -inch steam-pipe and operated by a 9-horse-power boiler, is 180 gallons per minute when the lift is as much as 25 feet; and for the 6-inch discharge, with a $1\frac{1}{2}$ -inch steam-pipe and operated by a 35-horse-power boiler, 1,000 gallons for the same lift.

The pulsometer is remarkably smooth in operation, and except for the slight click of the ball and the discharge of water in a steady stream, one would scarcely know it was pumping. Where a good-sized hoisting-engine boiler is in use on foundation work, it can be used to supply the steam for pumping. The

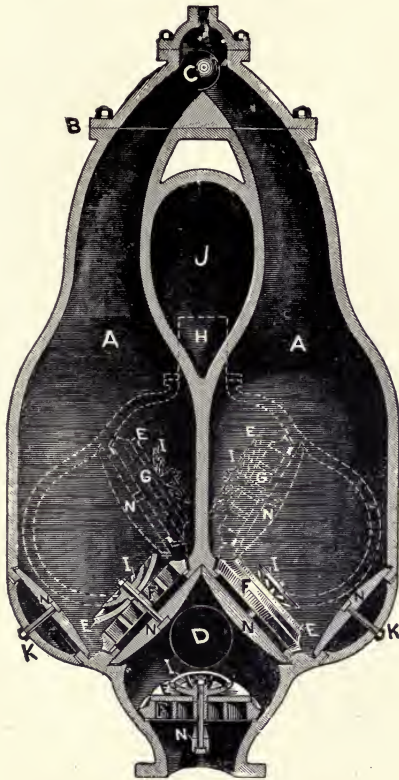


FIG. 87.—SECTION OF PULSOMETER.

work illustrated in Fig. 4 was easily kept free of water by a small pulsometer, while its use has been cited in a number of cases where the coffer-dam was pumped out by a centrifugal pump, and then the leakage kept under control by a medium-sized pulsometer, which required but little attention. The pump should be provided with a strainer at the bottom of the suction-pipe, all the connections must be air-tight, no sharp bends should be made

in the pipe, and with dry steam successful working will result. Another pump of similar construction is the Maslin automatic vacuum-pump, which differs from it in important details. What has been said regarding the pulsometer will apply as well to the Maslin pump.

All the foregoing devices are for use where the amount of water to be handled in a given time is of limited amount, but where large quantities are to be pumped out of coffer-dams in short periods of time, resource must be had to centrifugal pumps, which have reached a high state of perfection. Where the water is to be lifted 10 feet an ordinary reciprocating pump would exhibit an efficiency of only 30 per cent., while a centrifugal pump would have an efficiency of 64 per cent. For a lift of 17 feet the reciprocating type would have an efficiency of 50 per cent., while the centrifugal would reach its maximum of 69 per cent. efficiency, dropping to only 50 per cent. for a lift of 50 feet, while the other types would increase to 75 per cent. From this it will be seen that the centrifugal pump is essentially a low-lift machine.

Actual tests of pumps show that the maximum results are very seldom realized, a 9-inch discharge of one make showing an increase from 46.52 per cent. for a 12.25-foot lift to 57.57 per cent. for a 13.08-foot lift, while another make of 10-inch discharge shows a decrease from 64.5 per cent. for a 12.33-foot lift to 55.72 per cent. for a 13-foot lift. The greatest efficiency at hand is shown by a German pump with a $9\frac{1}{4}$ -inch discharge, a 10.3-inch suction and a 20.5-inch disk, running at 500 revolutions. The lift was 16.46 feet and the efficiency 73.1 per cent.!

That such results are not realized on actual work is readily understood when it is considered what little care is used to properly place and operate such a plant, how little attention is paid to having a proper boiler and engine, and what lack of care there often is to keep the plant in good repair.

An ideal outfit for operating by steam is shown in Fig. 88, where the engine is directly connected to a Heald & Sisco pump, all the trouble and vexation from the use of a belt being done

away with, and no loss of power through slipping of belts. The machine can be placed on the barge which carries the boiler, the suction-pipe being run horizontally across as in Fig. 88, while a short discharge-pipe discharges directly into the river. Where electric power plants are available a still better arrangement will

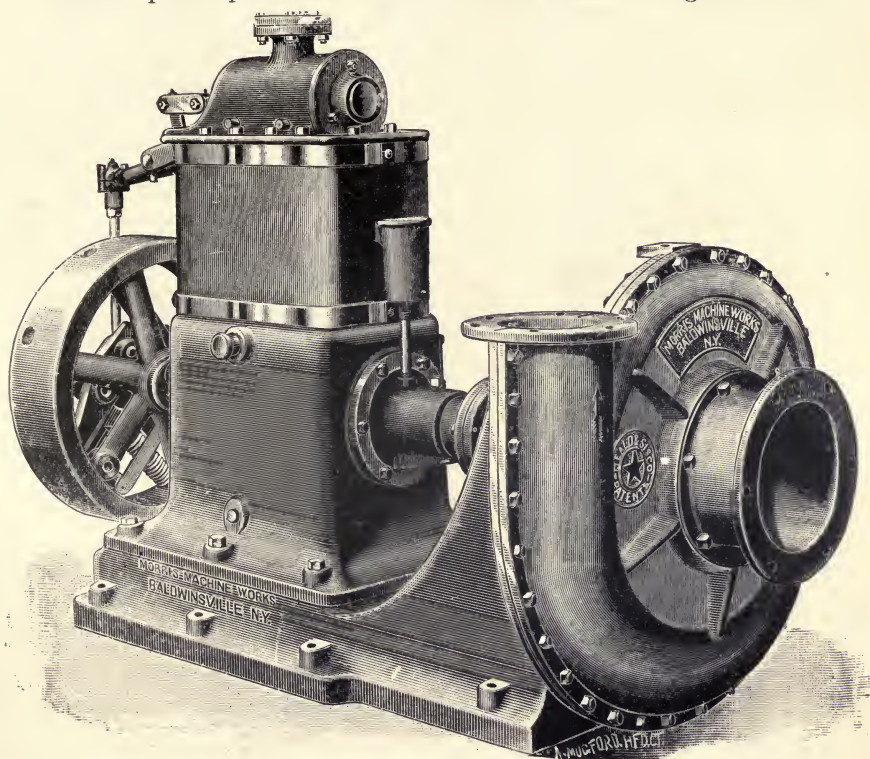


FIG. 88.—CENTRIFUGAL PUMP, DIRECTLY CONNECTED TO ENGINE.

be to have an electric motor directly connected to the pump, and all the trouble incident to the use of a boiler on the work will be avoided.

Electric power can also be used for hoisting and for pile-driving. Examples of the use of motors on hoisting machinery will be given in a later article.

The suction should always be fitted with a section of smooth-bore rubber hose (Fig. 89, *a*) to give it flexibility, a length of about

8 feet being usually sufficient. The best hose is made with a spiral metal core, which adds to its strength and durability.

The suction-pipe is ordinarily made of sections of wrought-iron pipe, with screw connections, but as this is troublesome to change sections, it will be found advantageous to use the spiral-riveted pipe with flange couplings (Fig. 89, *b*), and to have extra sections from 2 to 6 feet long, with several sections of each shorter length, so that the length of the suction-pipe can be readily changed

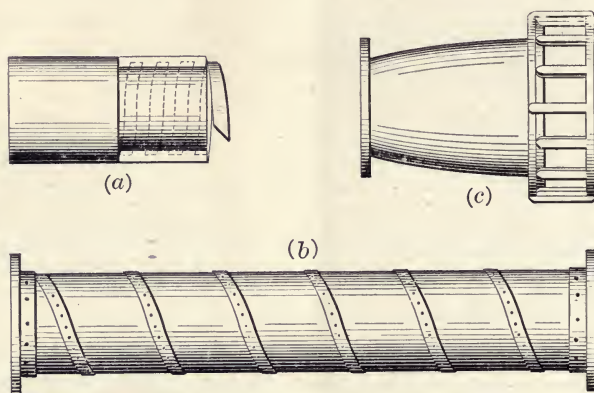


FIG. 89.—SUCTION DETAILS FOR PUMPS.

to suit the depth of the excavation. The flanges must be provided with rubber gaskets to keep the pipe air-tight.

The strainer (Fig. 89, *c*) is used to prevent large stones, sticks, or obstructions from entering and clogging ordinary pumps, and usually comprises a foot-valve to retain a pipe full of water and make the priming easy. The strainer or end of the suction-pipe is usually placed in the lowest point, and sometimes a box or sump is provided, as a well into which the water is drained from the other and higher portions of the work. A small set of falls should be attached to the foot to raise the pipe and clean out the strainer when necessary.

The centrifugal pump itself must be in first-class repair to do economical work, and should be a large enough size so that it need not be run beyond its economical capacity. The style of

pump to use will depend upon the work to be done, but for coffer-dam work a vertical pump could not be used easily and need not be considered. Where practically clean water is to be pumped an ordinary style of pump should be used, but where much mud or sand will be drawn up a sand-pump is best; and where a large part of the excavation is to be done with the pump, as at Topeka, a dredging-pump will be the proper type.

The pumping required on the Chattanooga work, 5,000 gallons per minute to a height of about 15 feet, would have been done most economically by a 15-inch pump, with a 40-horse-power engine and a 50-horse-power boiler. But a pump of this size would not find ready use in a contractor's work, and for this reason two 8-inch pumps would have been the better outfit to purchase, unless the work was very extensive; and each pump should be provided with a 25- or 30-horse-power engine, so as to run the pumps somewhat beyond the economical capacity, which could readily be done with a direct-connected engine, where there would be no belt to slip.

The work required on the Forth Bridge coffer-dams could also be done by the 15-inch pump above described, the lift being about 3 feet at the start and reaching 18 feet as the dam was cleared, the 340,000 gallons being pumped out in about one hour.

Centrifugal pumps are rarely required for a lift of over 20 feet on this class of work, which is only slightly beyond the economical lift, and the height should never exceed 30 feet, which would require for the 15-inch pump an engine of 75 horse-power.

The pump may be located on the coffer-dam, but in case of high water during the progress of the work the outfit may be damaged and it is best to place the pump on a boat, as in Fig. 56, with a section of horizontal suction-pipe across to the work, which should be as short as possible.

The ordinary type of pump (Fig. 88) may be fitted with a primer, consisting of a small, hand force-pump attached to one side of the pump, for filling the pump and suction-pipe. A more simple way is to provide a barrel above the pump, which can be kept full by using a small steam-jet, and, by means of a pipe with valve

from the bottom of the barrel to the top of pump, the contents can be emptied into the pump to prime it. Priming may also be easily

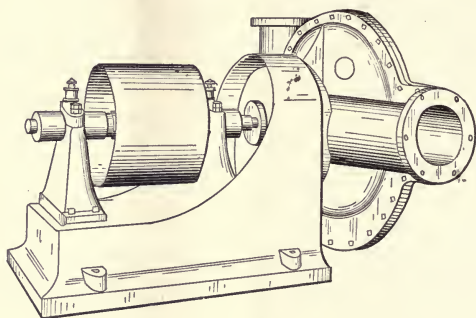


FIG. 90.—CENTRIFUGAL PUMP, DOUBLE SUCTION.

accomplished by inserting a hose into the discharge-pipe and filling the pump directly with a steam-jet.

Double suction-pumps (Fig. 90) allow the water to enter on

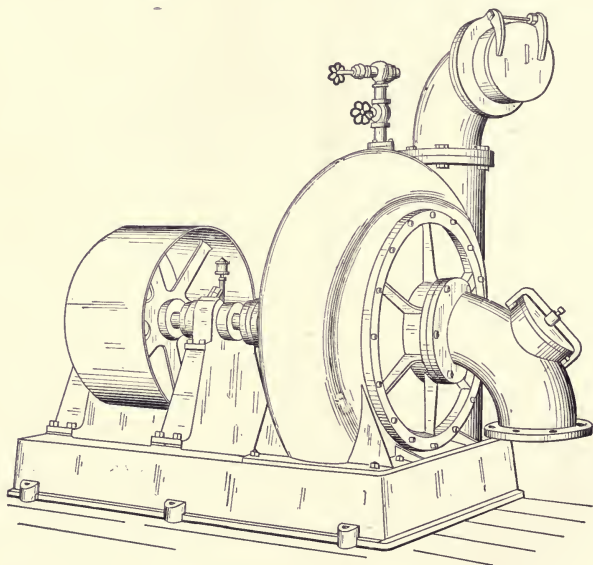


FIG. 91.—DREDGING-PUMP.

each side of the piston and thus a perfect balance is secured, which does away with all end-thrust on the bearings. This pump is most

easily primed by using an ejector, or a flap-valve such as is shown on the discharge-pipe of the dredging-pump (Fig. 91) and which serves to retain the water in the pump. Where a long discharge-pipe is to be used, a quick-closing gate-valve may be introduced into the pipe near the pump.

Where the material to be dredged out at the foundation site is mud or sand or partly gravel, it can be removed during the process of pumping by using a dredging-pump. In case there were 700 yards of material to be removed and an 8-inch pump was provided it would not be advisable to count on more than 10 per cent. of solid matter being discharged by the pump, as the suction could not be kept working close up to the sand or mud. By using a 30-horse-power engine, a discharge of 2,000 gallons per minute would be reached, or with 10 per cent. of loose solid matter the excavation would be made in less than two working days.

The piston of a dredging-pump (Fig. 92) is provided with large openings to receive the material, and the one illustrated is provided with side plates so that all wear is taken off the pump-casing.

One of the most remarkable pieces of work done with this class of pumps was the use of Edwards' cataract pumps in dredging the ship channel in New York harbor. This is described in the *Trans. Am. Soc. C. E.*, Vol. 25. The work was done by three dredges, which were much the same as small sea-going vessels, the largest being the *Reliance*, 157 feet long, and carrying 650 cubic yards of dredged material. Two separate pumps were provided, each with 18-inch suction-pipes reaching from the sides of

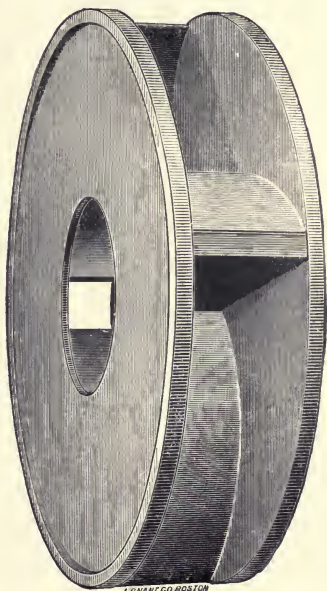


FIG. 92.—DREDGING-PUMP
PISTON.

the vessel and parallel to it down to the bottom to be dredged, being supported by suitable hoisting-tackle. These boats were kept under headway toward the dumping-ground while the dredging was in process. The average load during about a month's working of the *Reliance* was 585 cubic yards and the average time of loading about 48 minutes, while the average number of loads per day was 6.73.

These dredges removed the enormous quantity of 4,299,858

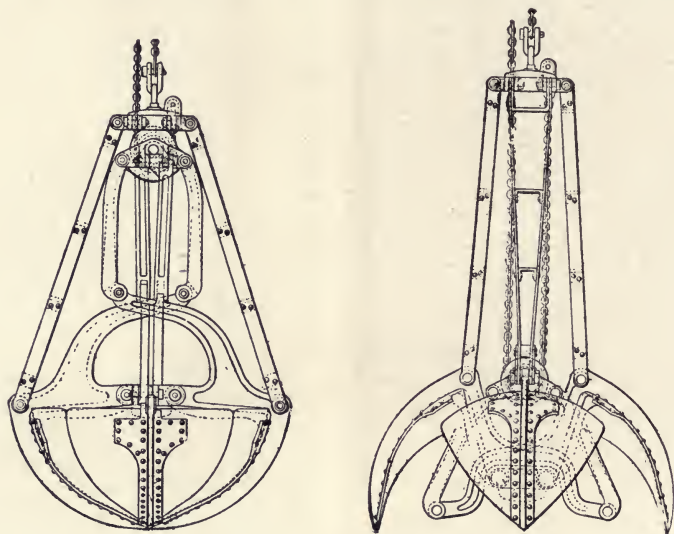


FIG. 93.—LANCASTER GRAPPLE.

cubic yards of material at an average price of 24.48 cents per yard, the lowest price being about 17 cents, the average price paid for other forms of dredging being 40.53 cents. On foundation work the amounts to be removed would be small and the cost for this reason much higher, yet owing to the smaller cost of the plant that would be required the cost need not be greatly in excess of the above. It is usual, however, as the amount to be dredged will cost such a small proportion of the total cost of the substructure, to figure from \$1 to \$2 per yard for excavation in ordinary coffer-dams.

Reference has already been made to hand dredging, and a

very cheap and effective scraper was illustrated in Fig. 8. Where dredging is to be done in tubes, wells, or puddle-chambers, it can be done by a clam-shell dredge or grapple such as was shown in Fig. 57, in use on the Hawkesbury foundations.

The Lancaster dredge (Fig. 93) is a well-known form of this type of machine, and can be operated from an ordinary derrick



FIG. 94.—SAND-DIGGER.

which is served by a double-drum hoisting-engine. This dredge will work best of course, where there is some depth of soft material to be removed. While a large dredge would generally be hired by a contractor, these buckets can be owned by him and the work carried on cheaply and conveniently.

Sand-diggers such as were mentioned in Chapter II can often be hired where other means are not at hand, or they can be rigged up very cheaply if necessary. A very simple one (Fig. 94) can be built on an ordinary barge, the engine being an ordinary one with a vertical boiler, while the buckets are mounted in a very

simple manner and operated through a well in the center of the boat. Such a dredge will dig about 100 yards of sand per day, with only two men to attend it, and will use less than one-half ton of cheap coal, the total cost per yard thus running below five cents. Large elevator dredges of this type are very elaborate affairs, and as they are in wide use they can often be hired for making excavations.

The best-known form of dredge, perhaps, is the dipper dredge. The Osgood machines (Figs. 95 and 96) in use on the New York State canals are among the best machines of this kind in use. Such dredges are more simple in construction than elevator machines, and are consequently easier and cheaper to keep in repair. The hull is $70' \times 17' \times 6'$ with two 6-foot pontoons, which are removed when going through locks. The engines consist of a double-drum main engine with $8'' \times 10''$ cylinders, a swinging-engine with $6'' \times 8''$ cylinders, and a crowding-engine, $5'' \times 6''$ cylinders, which are all used in operating the digger of $1\frac{1}{4}$ yards capacity on a steel boom 45 feet in length.

The crowding-engine is used to control the dipper and enables it to make a practically level bottom at one cut, and also thrusts the dipper far enough beyond the boom to allow it to dump 52 feet from the center. This dredge, which cost complete \$10,000, is operated by a crew of only four men and consumes but one ton of coal per day of twelve hours, the average excavation during four months' work being 549 cubic yards per day. The machine has sufficient power to dig hard-pan, bowlders, and very soft shale rock.

A dredge of this make, of $3\frac{1}{2}$ yards capacity, working in mud and sand, has dug material at the very low actual cost of .99 of one cent! This, of course, was an exceptional case, and the cost will rarely fall below five cents per yard on easy work at a depth not exceeding 10 feet, and in such small amounts as would have to be dredged on coffer-dam work and in about 20 feet of water the actual cost would likely reach fifteen cents per yard. In case the dredge should be hired to do the work, a charge of from twenty to thirty cents per yard would not be excessive, depending, of course on the class of material and the amount.



FIG. 95.—OSGOOD DIPPER DREDGE, NEW YORK STATE CANALS.

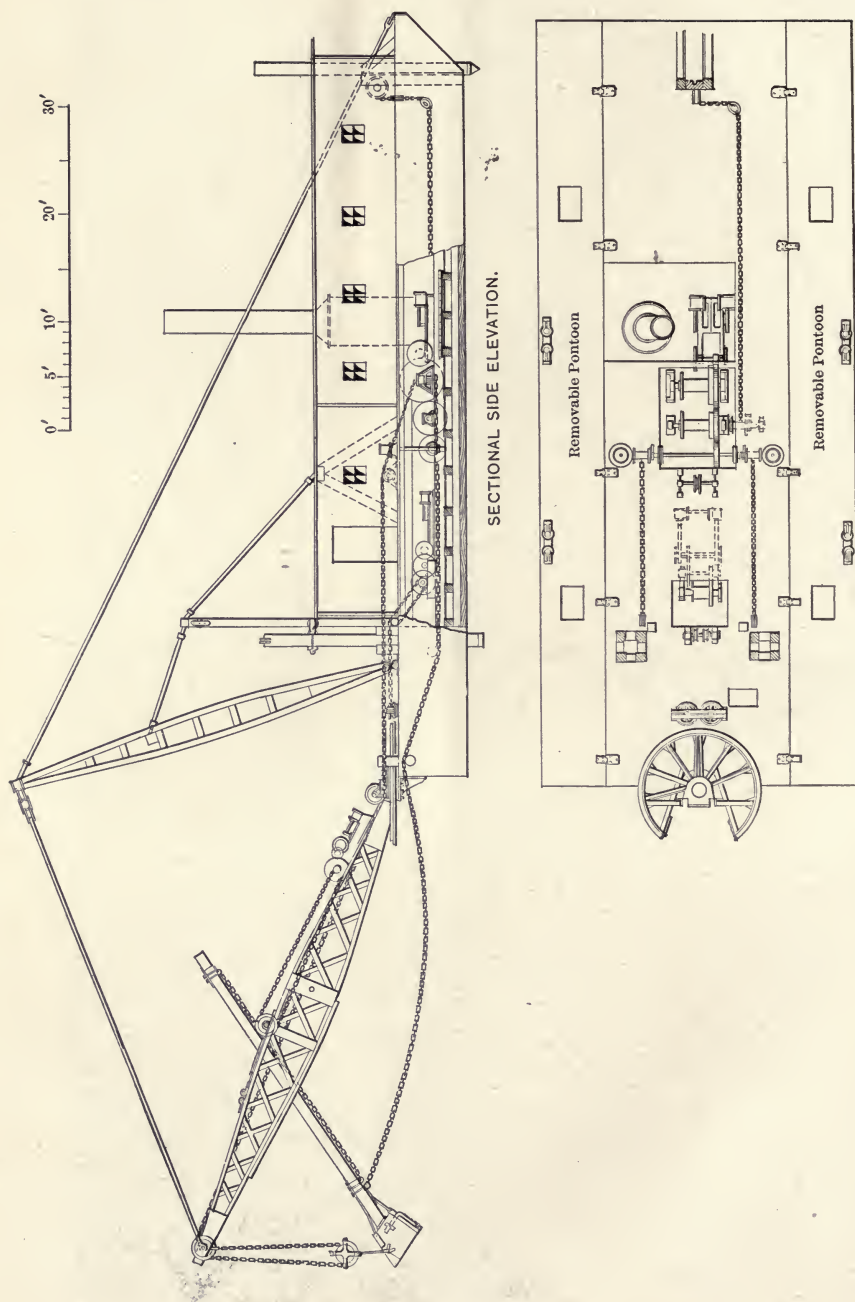


FIG. 96.—OSGOOD DIPPER DREDGE, DETAILS, NEW YORK STATE CANALS.

CHAPTER X.

THE FOUNDATION.

THE coffer-dam is only the means of reaching a desired end, and this must be borne in mind and the construction made as simply as possible to obtain a first-class foundation.

When the coffer-dam is completed and pumped out, work can then proceed if the pumps are able to control the water easily. The character of the foundation having been previously decided upon, after a careful examination of the site, it is assumed that the temporary work has been executed in a manner which is properly related to the permanent structure.

The different kinds of bottom likely to be encountered are: First, light sand and gravel or mud of unknown depth; second, similar material overlying either cemented gravel, clay, hard-pan, or rock; third, a clean rock bottom, which is approximately smooth and level; fourth, a sloping rock bottom, which is either smooth or rough, and fifth, a rough and irregular rock bottom.

Should the bottom be of the first kind—light sand and gravel or mud of unknown depth—the soft upper layer may have been removed by a dredge previous to the building of the dam, or it may be removed by a dredge or grapple from within the inclosed area, and without the necessity of keeping the dam pumped out, or pumping may be kept up with a dredging-pump and the light material removed in this way, after which the heavier material may be removed as deep as necessary by hand-shoveling and a dirt-box, as shown in Fig. 56. In such a bottom the foundation is usually made by driving piles from 2 to 4 feet centers, this distance being regulated by the bearing power, as determined from

Wellington's formula in Chapter IV, and building upon the tops of the piles, after they have been cut off to a level below low water, a grillage of timber. The space between the piles should be filled with broken stone or concrete, and the grillage placed entirely below low water, the coffer-dam being kept pumped out to allow this work to be done, and also during the laying of the footing courses of the masonry which are below the water.

When the soft bottom overlays good clay, hard-pan, or rock, as in the second case, and the depth exceeds 20 or 25 feet below the water-surface, piles may be driven to the harder substratum and act as bearing-piles. But when the depth is in the region of 20 feet or less, it is best to excavate and place the foundation masonry directly upon the solid bottom. The foundation will be of the character described for some of the following cases:

The third class is similar to the foundation at Chattanooga after the gravel was removed. The fissures in the rock are filled up or closed with cement and concrete, and a leveling course of concrete put down on which to found the pier (Fig. 49).

Bottoms of the fourth class should have all the loose and decomposed rock removed and steps cut out by blasting and wedging, to give a secure hold for the foundation, but if it is simply rough and irregular a leveling course of concrete will be all that is required on which to start the pier. Bottoms of clay and hard-pan will require a very similar treatment, except that the leveling course of concrete must be made of sufficient thickness to properly distribute the pressure, which will seldom be less than 3 feet and can often be increased with advantage to 6 or 8 feet. An example of the stepping of rock bottom was given in the account of the Forth Bridge piers in Chapter VII and was shown by the dotted lines in Fig. 64.

Where there is a current caused by leakage through the sides of a coffer-dam, or from the bottom, or if the water within the dam is agitated by the pumping, it will be best, after the bottom is clean and properly prepared, to allow the water to run in and then deposit the concrete through the still water. This has been successfully accomplished through 25 or 30 feet of water, and

while some engineers recommend allowing the concrete to set from one to three hours before depositing, to prevent the cement from washing out of the concrete, this is not necessary nor advisable if the proper care is exercised and the proper apparatus used. The concrete should be made from one-third to one-half richer than would be used for similar open-air work, as there will be some loss of strength.

The simplest method is to deposit the concrete in paper sacks by sliding them down a smooth wooden or iron chute, or by loading them into a box or skip and dumping them out after the box reaches the bottom. The sacks should be of tough paper, similar to flour sacks, and when they reach the bottom they may be broken by a pike-pole and the concrete allowed to run together. Thin cloth sacks are sometimes used and they become fairly well cemented together by the mortar which oozes through.

Where the amount of concrete is considerable it will be best to use a tube or bottom dumping-box. For placing concrete under water on the Boucicault Bridge over the river Saône in France a wooden tube 16 inches square was used. This is described in the *Engineering News* of May 18, 1893. The tube was carried transversely across the caisson on a traveling-crane which ran lengthwise of the caisson on tracks on the sides, thus allowing the tube to be moved in any desired direction. The tube was built in sections which could be easily removed, was provided with a hopper at the top into which the concrete was dumped, and a drop-door at the bottom to let out the concrete. The tube was filled as it was lowered down into the water, and opened when within 16 inches of the bottom. As concrete was dumped in above, the tube was moved about and a 16-inch layer of concrete deposited. When one layer was complete, another of the same thickness was deposited. This method of using 16-inch layers was said to have obviated laitance or the exuding of the gelatinous fluid which prevents uniform setting. The concrete was deposited about the heads of the piles and no grillage used. The thickness of the concrete, which was deposited at the rate of from 90 to 100 yards per day, was 9.84 feet, and was allowed to set fourteen days before the pier was begun.

A metal tube may be used, such as was employed on the Harvard Bridge at Boston by W. H. Ward. This tube (Fig. 97) was not provided with a bottom and the first filling of the tube was consequently done after the tube was lowered and the concrete became somewhat washed. This may easily be prevented by using concrete in paper sacks to fill the tube the first time. The tube was suspended from a derrick and was moved about so as to keep the concrete level and deposit it in layers. This account is taken from Vol. 31 of the *Engineering News*, from which is taken the following description of a metal bucket used by W. D. Taylor on the Coosa River:

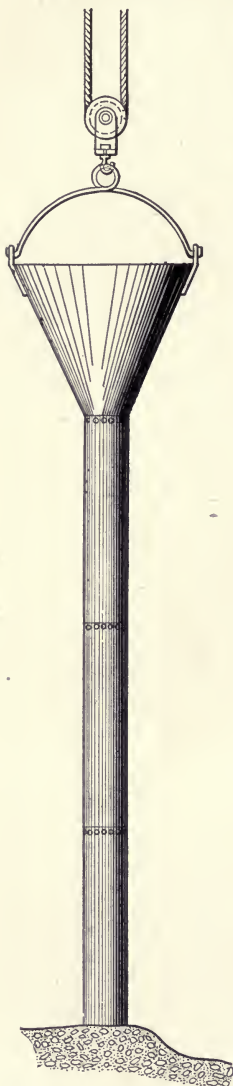


FIG. 97.—METAL TUBE
FOR CONCRETING.

This bucket (Fig. 98) was of riveted construction and held one yard of concrete. The maximum depth of water was 26 feet, at which depth the bucket and its load became so lightened that the bucket tripped as soon as the flanges touched the bottom. Similar boxes are often constructed of wood, or they are often made "V"-shaped, one side being arranged to open and dump the load.

For concrete work of this character natural cement is often used, but on all important work Portland cement should be employed. The proportions range from one of cement, two of sand, and four of broken stone, to one of cement, three of sand, and six of broken stone.

On such a base either a masonry or monolithic concrete pier may be constructed. The pier at Little Rock (Fig. 52) was of

this construction and of the composition given in Chapter VI. A similar piece of work was constructed on the Red River Bridge on the St. Louis & San Francisco Railway and is described in

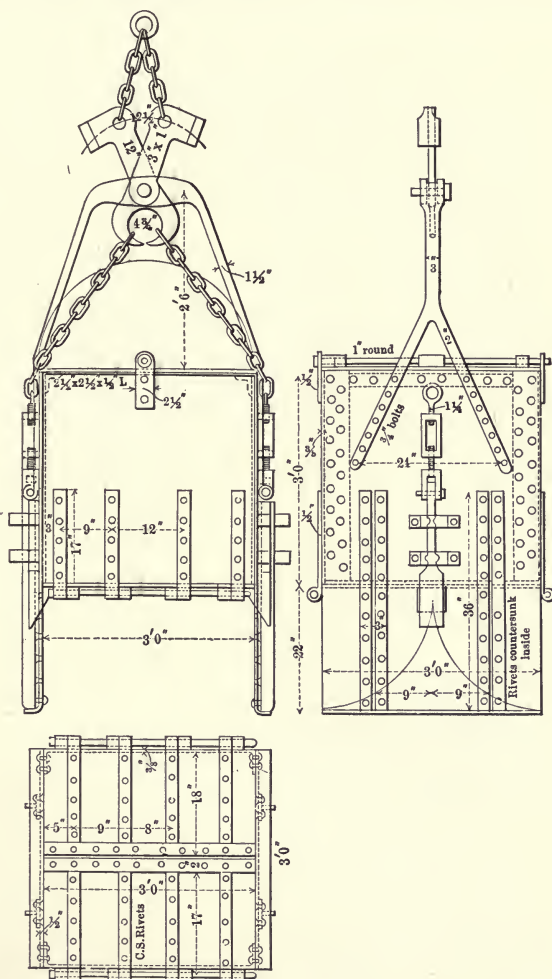


FIG. 98.—METAL BUCKET FOR CONCRETING.

the *Engineering News* of June 2, 1888, by C. D. Purdon, assistant engineer in charge of the work under James Dun, chief engineer. The cribs were filled with Louisville cement concrete up to within 2 feet of low water, on which was built the pier. (Figs. 99 and 100.)

"After the crib had been filled with concrete and the surface leveled off, the center lines of the pier were located and a frame of 2"×8" plank, the shape of the pier, and 4 inches larger to allow for lagging, was placed in exact position and held by pieces

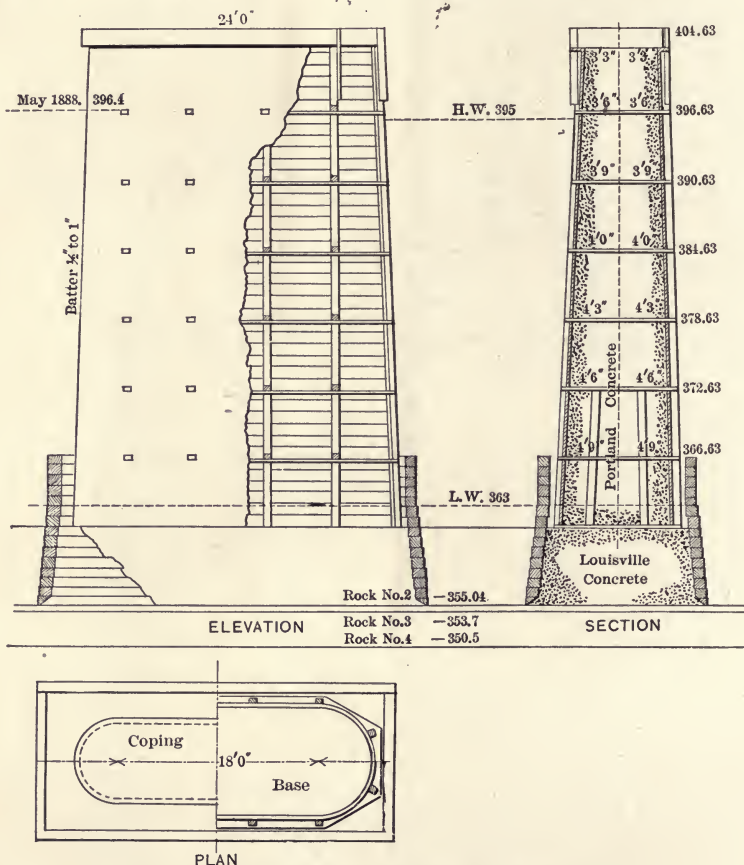


FIG. 99.—CONCRETE PIERS, RED RIVER BRIDGE.

spiked to the crib. On this frame upright posts 6"×6" and 5 feet 10 inches high, with a batter of 1/2 inch per 3 feet, were set in the position shown on the drawing, then the feet spiked to the frame and another frame similar to the first, but 6 inches narrower, placed on them. This again was brought to exact position and braced to the crib and the frame completed by putting lagging of 2-inch

plank inside the posts and spiking to them. This lagging was horizontal in the body of the pier and vertical ($2'' \times 4''$) at the ends, beveled pieces being introduced in the ends at intervals to make up the difference of the upper and lower circles. Next $2'' \times 6''$ planks were placed across on the tops of the posts, running

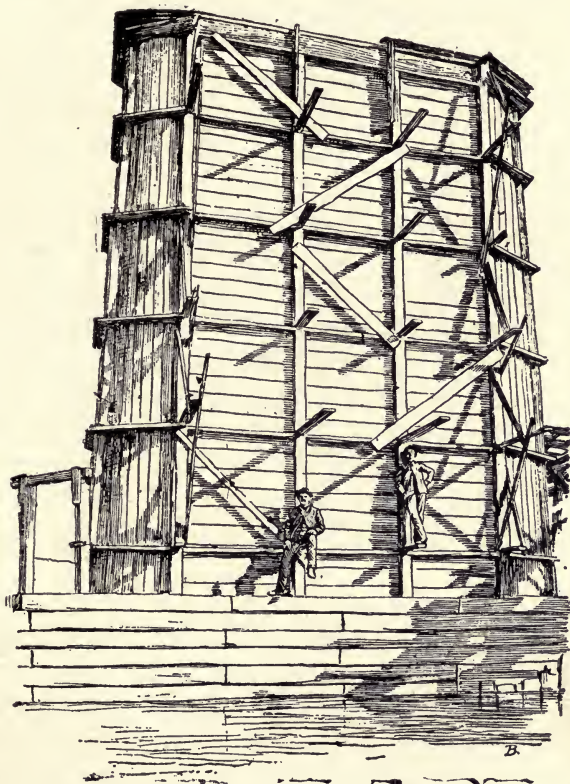


FIG. 100.—CONCRETE FORMS, RED RIVER BRIDGE.

clear through the pier, to act as braces. In the rest of the frames these braces were allowed to extend about 6 feet on each side and the frame braced by spiking plank to them and to the vertical posts. After a section of frames was completed a bed of cement mortar about 2 inches thick was spread all over the concrete in the crib. On this rough stone, in such pieces as one man could easily handle, was placed so that no two pieces would be

closer than 2 inches, nor any piece within 2 inches of the frame, the stone being thoroughly wet before laying.

"Next, on this course of stone another bed of mortar was placed, sufficient to fill all the spaces between the stones and remain about 2 inches thick above them. It was then well rammed with rammers made by inserting a handle in a section of a pile, except at the edges, where a rammer made of a 2-inch plank cut in the shape of a spade was used, to insure a perfect skin of cement without any breaks. After this had been well rammed, another layer of stone was placed and covered with mortar as before, and so on.

"The coping, which was made similar to the body of the pier, was finished by about $1\frac{1}{2}$ inches of cement mixed with sand one to one, fluid enough to be struck off by a straight-edge, the top of the frame being dressed and leveled for that purpose.

"After the pier had been completed the frames were removed and the braces running through the piers cut off by a chisel inside the concrete. Then, to make a smooth surface, the pier was thoroughly wet and plastered with a mixture of one part sand to one part cement, after all the rough or loose portions had been scraped off. This was mainly done for appearance."

The mortar for the body of the pier was made of one part Alsen's German Portland cement and four parts of sand. There was used about $1\frac{1}{3}$ barrels of cement to a cubic yard of completed pier. In mixing the mortar eleven ordinary pails full of water were used to one barrel of cement, which caused the water to just appear on the surface when the tamping was done.

The lock walls on the Illinois and Mississippi canal have been constructed of monolithic concrete under Captain W. L. Marshall, Corps of Engineers. The work was executed under L. L. Wheeler, engineer in charge, from whose account, in the report of the Chief of Engineers for 1894, the following is taken:

"The rules adopted for the work were adhered to and are worthy of careful study.

"I. The forms or molds of the walls will be divided by ver-

tical partitions perpendicular to the longest axis of the mass, and the walls be constructed by filling alternate sections.

"II. The sections will be filled in horizontal layers, well rammed, each layer to be deposited before the 'initial set' of the previously deposited layer. When the work of filling a section is begun it must proceed without intermission to completion, working night and day if necessary.

"III. The facing and backing must go on simultaneously in the same horizontal layers, using the same cement in the facing as in the backing, with no defined lines of demarkation between the facing which contains no stone and the concrete backing.

"IV. When the top surface of the coping is reached it will be finished after ramming by cutting off the excess by a straight-edge, and rubbed smooth and hard by a float. No plastering or wet finishing will be allowed.

"V. The facing of the walls will not be finished by plastering or washing with cement after the forms are removed, nor dressed in any manner beyond chiseling away rough ridges, should the plank forming not be smooth.

"VI. The concrete shall be mixed with all the water it will take, without water showing after ramming, or without 'quaking' upon ramming.

"VII. At such intervals as may be necessary vertical wells, at least 1 foot square, will be formed along the middle of the masses of concrete, reaching to near the bottom thereof. The masses of concrete after forming will be kept sheltered from the sun, the outer surfaces kept moist and the wells kept filled with water until well set, or about three weeks. The walls will then be filled with concrete.

"VIII. In preparing the cement for mixing with other ingredients of concrete, from five to ten barrels will be kept thoroughly mixed dry, to guard against chance barrels of defective cement, and the necessary quantity of cement will be taken for each batch from this mixture.

"IX. Two cements of different qualities shall not be used in the same section, but as far as practicable each mass shall

be homogeneous throughout, but a slight excess of cement in the facing to reduce its capacity to absorb water."

The rate at which the concrete was deposited in the work was determined by the rate of ramming, and but one yard every five minutes was deposited. The forms (Fig. 101) were lined with dressed pine plank 4"×8" on the face, of uniform thickness, and with 2-inch rough plank on the back.

Rough plank is sometimes used on such work and lined with oiled paper, or ordinary dressed plank may be used and coated with soft soap. In most sections of the country crushed broken

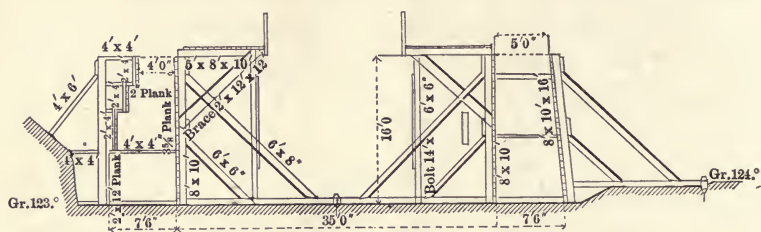


FIG. 101.—CONCRETE FORMS, ILLINOIS AND MICHIGAN CANAL.

stone can be obtained, but owing to the magnitude of this work a crusher was built (Fig. 102) and was found to work very satisfactorily. The concrete mixer shown in Fig. 102 was operated by a 15-horse-power portable engine. The proportions finally adopted for the concrete were one of cement, three and one-third of gravel, and four of broken stone, while the facing and coping were composed of one part cement and two parts of clean river sand.

That the sand for concrete be clean and sharp is very essential, and any loam or dirt must be washed out. Equally essential is good, clean, sharp, broken stone without dust or dirt. The cement used on the above work was a German Portland, but several of the American Portlands are first-class and will give as good results as the imported.

Where good, fresh cement is being supplied, a few tests to a car-load will be sufficient, and for cements of the grade of Atlas or Empire the guarantee of the manufacturer, supplemented by a few tests, should be sufficient. But for cements which have been

shipped by water, tests should be made from every five or ten barrels.

The Atlas Cement Company recommend, for concrete laid in open air on moist ground where great weight must be carried, one of cement, two of clean sharp sand, and four of 2-inch broken

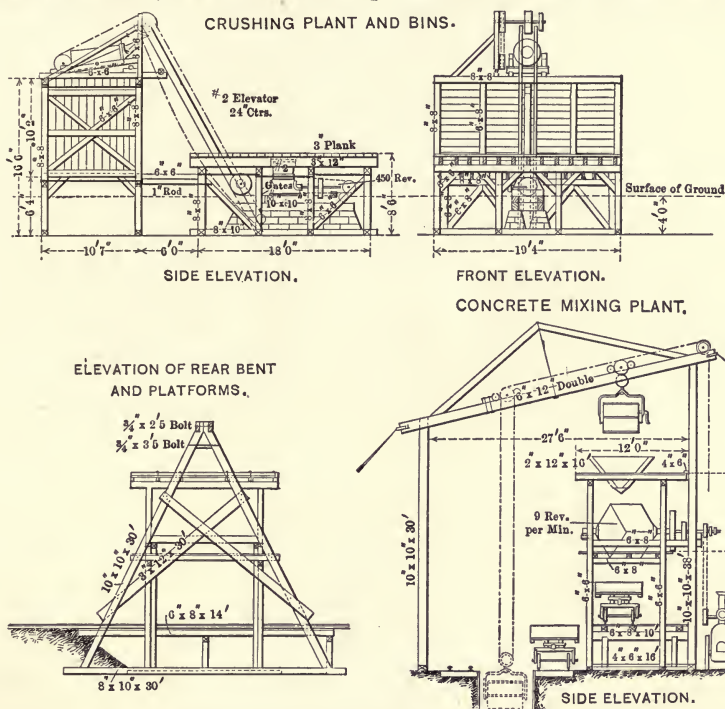


FIG. 102.—STONE CRUSHER AND CONCRETE MIXER, ILLINOIS AND MICHIGAN CANAL.

stone; this sand and cement to be thoroughly mixed dry, then just enough water added to thoroughly moisten, and the mass turned over at least twice, when the stone is to be added in a thoroughly wet condition. This must then be put at once into the molds and well rammed.

Where a solid bottom is to be built upon, the proportions of one of cement, three of sand, and six of broken stone are recommended. For ordinary construction one of cement, four of sand, and eight of broken stone, while to obtain a concrete as

strong as ordinary natural cement concrete, one of cement, five of sand, and ten of broken stone can be used.

The average cost of such concretes, including labor, tools, timber forms, and a fair profit to the contractor, would be for the first \$8 per yard, for the second \$7, for the third \$6, and for the fourth \$5.

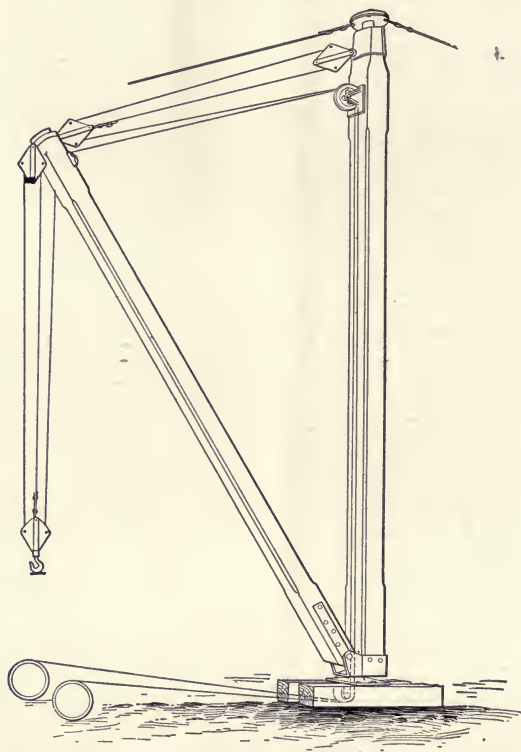


FIG. 103.—DOUBLE-DRUM GUY DERRICK, AMERICAN HOIST AND DERRICK CO.

Where the leveling course of concrete has been put in and the pier is to be of stone, the footing course should be of carefully selected material. They should be large stones with good beds, and should be as thick or preferably thicker than the courses above. Where the bearing pressure does not exceed two tons per square foot, the footing courses may be stepped by allowing each course to project about one and one-third

times its thickness, depending of course on the quality of the stone.

The usual way of handling the material for foundations and

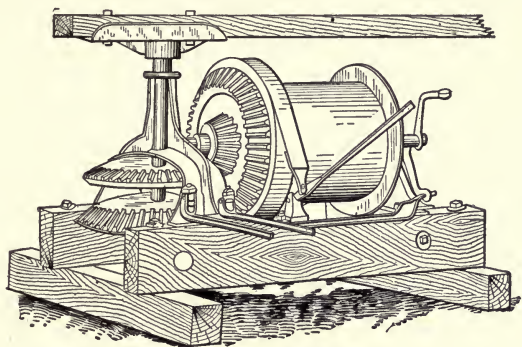


FIG. 104.—SINGLE-DRUM HORSE-POWER, CONTRACTORS' PLANT MFG. CO.

piers is to boat it to the site, where it is placed by a stiff-leg derrick, or, if guys can be used, by a derrick with wire-rope guys. The fittings for such derricks can be obtained from a number

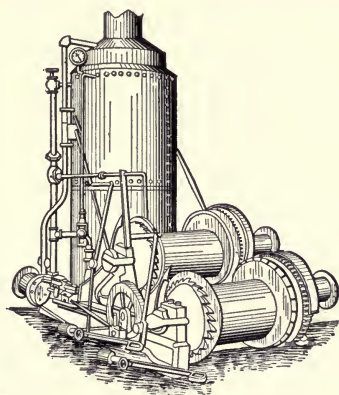


FIG. 105.—DOUBLE-DRUM HOIST-ENGINE, LIDGERWOOD MFG. CO.

of firms, an American Hoist and Derrick Company outfit being shown in Fig. 103. This is rigged to be operated by a double-drum hoist, which can be one operated by horse-power (Fig. 104) if the piers are near the bank and if steam-power is not available. The usual form, however, is a double-drum steam-hoist like the Lidgerwood machine shown in Fig. 105. Where electric power

is available an electric hoist (Fig. 106) should be used, as it will be found much more convenient.

Works of any magnitude should, however, be fitted from the beginning with a cableway, which will avoid the necessity of boating the materials, erecting of large derricks, and facilitate in every way the prosecution of the work, besides often making a balance on the right side of the ledger. The Lidgerwood cableway on dam No. 11 of the Great Kanawha River, a tower

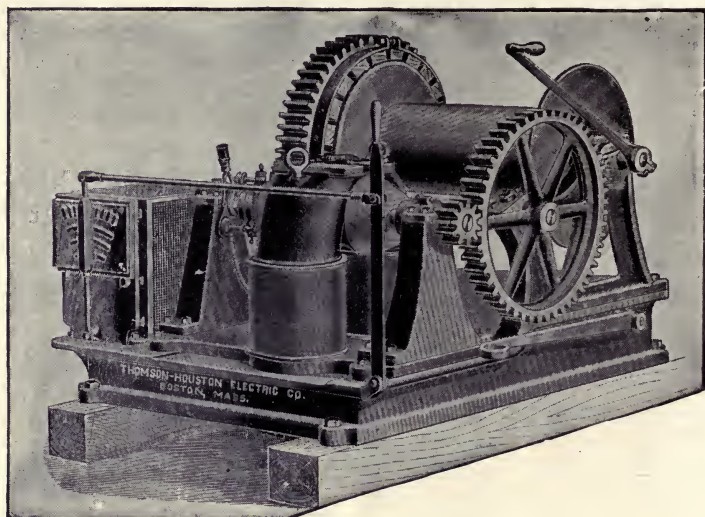


FIG. 106.—LIDGERWOOD ELECTRIC HOIST.

of which can be seen in Fig. 9, had a span of 1,505.5 feet and carried a net load of four tons on a main cable $2\frac{1}{2}$ inches in diameter. The stone quarry was located on one bank, and the stone was taken directly to the stone yard and to the work in the river. A seam of coal in the quarry also supplied fuel for the dredges and pumps, the coal being handled by the cableway, as was also the material from the railroad siding on the opposite bank.

The details of these cableways have been developed and perfected to a wonderful extent, as a result of their use on the Chicago drainage channel. The engine for operating one of these with a capacity of eight tons has double $10'' \times 12''$ cylinders,

the cranks being set at an angle of 90° and is provided with reversing link motion. The double drums regulate both the hoist at a speed of 300 feet per minute and the travel along the cable at 1,000 feet per minute. A 70-horse-power boiler is required.

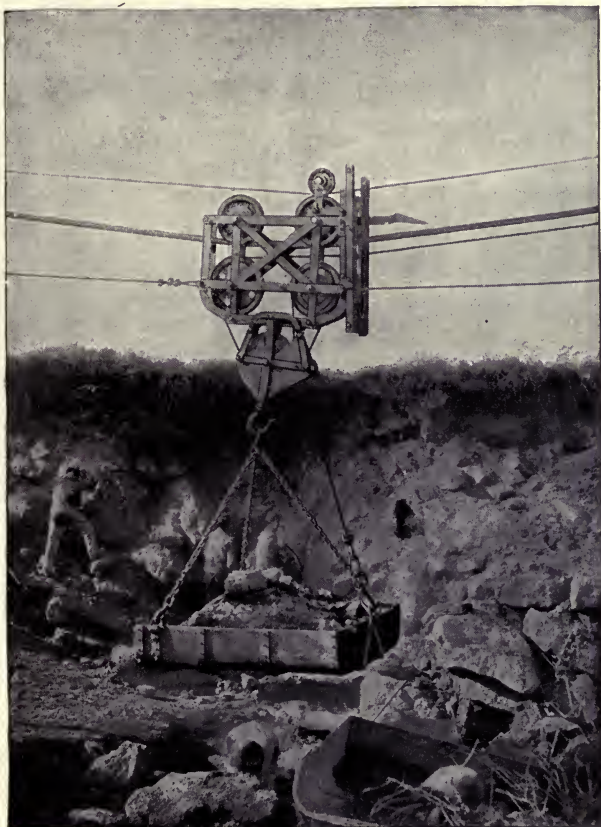


FIG. 107.—LIDGERWOOD CABLEWAY CARRIAGE AND SKIP.

The carriage and skip, which are automatic in action, are shown in Fig. 107, the capacity of those on the drainage channel being 1.8 yards, and the average of a month being about 600 yards per day of ten hours. The cost of operation, including labor, fuel, and everything except interest on plant and repairs, was less than \$18 per day or from three to four cents per yard.

The cableway on the Coosa dam and lock (Fig. 108) had a capacity of about eight tons and made a round trip on an

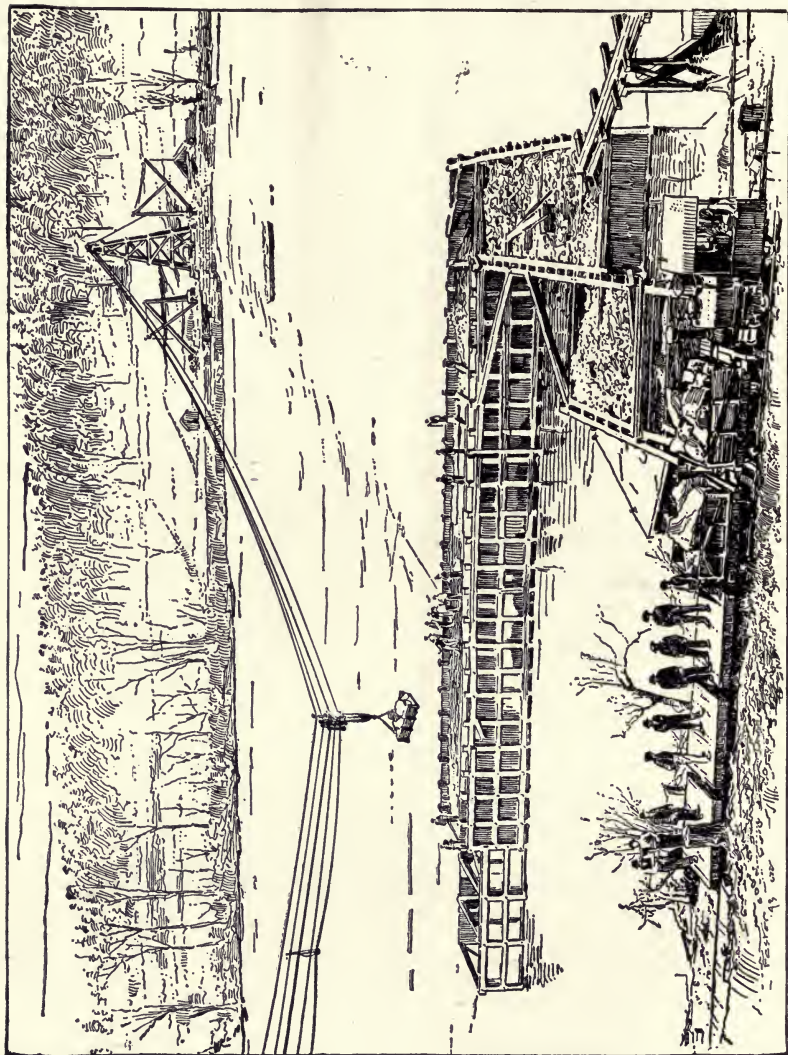


FIG. 108.—LIDGERWOOD CABLEWAY AT COOSA DAM. SPAN 1,012 FEET.

average of about three minutes. Such a plant is out of reach of high water and of trains where used over railroad tracks as at the North avenue bridge in Baltimore.

The Court street stone-arch bridge at Rochester, N. Y., of eight spans, was constructed with the aid of a cableway, which was also used to remove the old bridge and piers. A cableway of one span was used to construct the Melan concrete-arch bridge at Topeka, Kan. The bridge has five spans and a total length of 650 feet. During the extreme high water in the early part of 1897, when everything was completely inundated, and an ordinary derrick plant would have been swept away, the cableway was high and dry out of reach of the flood.

The prevailing low prices of contract work make it necessary to employ every improvement on important engineering work, and the cableway has doubtless come to stay as one of the most remarkable of our tools.

CHAPTER XI.

THE FOUNDATION (CONTINUED).

THE determination of the exact character of the foundation is entirely dependent upon the bearing capacity of the soil or foundation bed. In cases where there is large doubt as to what the bottom will carry per square foot of surface, it is always best to make some tests to arrive at definite conclusions; but in ordinary cases it is possible to adopt figures well within safe limits, and thus avoid the trouble and expense of experimenting. Unless the experiments are very carefully conducted, precedent is the better method.

For the State Capitol at Albany, N. Y., one of the most important structures in the United States, very careful and elaborate experiments were conducted by W. J. McAlpine, the engineer in charge of the work; and as the material, which was blue clay, was found to sustain a load of 6 tons per square foot, it was decided to adopt 2 tons as the safe load to put upon the foundation bed.

The Congressional Library at Washington, D. C. (Fig. 109), another very important building, had the foundation constructed to come within $2\frac{1}{2}$ tons per square foot, although the yellow clay was found to carry a total load of $13\frac{1}{2}$ tons per square foot.

It is always possible, of course, to thoroughly drain the foundations of a building, or to at least know the exact condition in which the foundation bed will exist; but for bridge work the circumstances are very different, and after the foundation is once in place, unless it be on solid rock, examinations are very difficult, so that it is necessary to be much more sure as to what the material will carry.



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FIG. 109.—CONGRESSIONAL LIBRARY, WASHINGTON, D. C.

The Bismarck Bridge (Fig. 110) across the Missouri River, on the line of the Northern Pacific Railway, has the piers (Fig. 111) founded upon the clay which was found to sustain a load of 15 tons per square foot before settlement ensued, and the actual load is 3 tons per square foot. From a report on this work the following account is taken of the character of the foundation bed:

“With the exception of some thin strata of soft sandstone of irregular thickness and extent, no rock is found in position in this part of Dakota. The entire country is underlaid with a very hard stratified clay, the depth of which has not been ascertained. Borings proved this clay to be at least 100 feet thick on the line of the bridge, and a hole intended for an artesian well has since been sunk within the Bismarck city limits to a depth of over 1,300 feet in the clay. This clay, however, is in many respects more like a rock than a clay; small specimens tested for compression have sustained a weight of over 300 pounds per square inch without crushing, and when they gave way yielded like rock, and showed no tendency to bulge out at the sides. Water has little or no effect upon this clay, even where the current is extremely strong, but when exposed to the dry air the clay slakes rapidly and crumbles to pieces.”

The Roebling Suspension Bridge at Cincinnati, Ohio, has the towers (Fig. 112) founded on coarse gravel, although by going 12 feet deeper solid limestone could have been reached. The load of 3.63 tons per square foot adopted was, however, well within safe limits for so large a spread as the base of the towers.

“No definite plan of foundation had been fixed upon; none could be safely adopted before the excavations had been sunk some considerable depth. Whether a solid rock bottom could be reached on either side was uncertain; so also was the question of piling or solid layers of timber left open. It is known that the bed of the Ohio River is throughout its extent underlaid with rock, at no great distance below the surface. But the depths at which the rock is found varies very considerably. At all the rapids and shallows the rock forms the bed itself; in the pools between, heavy layers of gravel, sand, or mud are deposited. At or near



FIG. 110.—BISMARCK BRIDGE, NORTHERN PACIFIC RAILWAY.

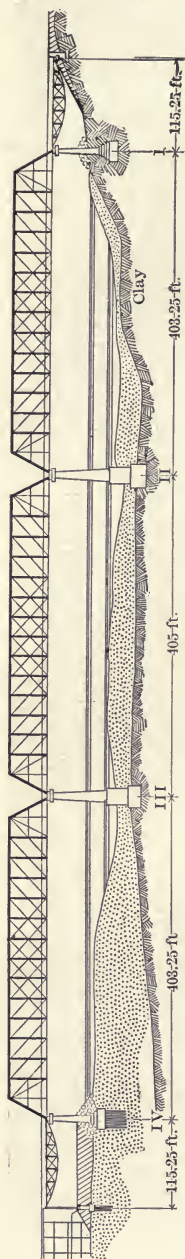


FIG. III.—BISMARCK BRIDGE FOUNDATIONS.

the mouths of tributaries, the rock has generally been excavated by the action of the water to a great depth; the soft material has taken its place, and consequently good foundations can only be made by heavy expenditures. At the site of the bridge, a bed of blue limestone and shale, not very solid, underlies the river bed at a depth of about 12 feet below the lowest part of the channel. A short distance above the bridge this shale is laid bare at low water on the Covington side, a part of it extending half way across. Under the Covington tower, a heavy bed of coarse sand, mixed with gravel, is found above the rock, while the surface layer is composed of the original clay bottom which forms the river banks. On the Cincinnati side, the original clay or loamy bottom has, to some extent, been washed away, and latterly been filled up again by the materials obtained from cellar excavations.

"In this artificial bank the excavation of the Cincinnati tower was commenced about the 1st of September, 1856, and sunk down to the level of the river, which, during this and the next two months, fell to low-water mark. A little rise of 4 feet intervened, but the river fell again, and continued low until the month of December. It was owing to this remarkable favorable state of the river that we so well succeeded in our foundations, and at a cost which must be considered as very small, considering their magnitude and the sudden floods which may occur at almost any time and sweep over and destroy costly preparations.

“By a wise resolution of the Board of Managers, the work was not to be commenced before a bona fide cash subscription of \$300,000 had been secured. Contrary to my expectations, this



FIG. 112.—CINCINNATI SUSPENSION BRIDGE.

subscription was rapidly obtained; and in view of the promising state of the river, it was concluded to forthwith commence the foundation work. But no preparations whatever had been made, no materials on hand, no machinery, and no efficient pumps. The total want of the latter proved a very serious drawback, and seriously threatened to defeat the enterprise at the very outset.

It is true, the city was full of steamboat-pumps, but of such small dimensions and such construction that they were of no account in such an operation. Raising clean water is an easy process, but to raise large masses of soft mud and sand is not so easy. After experimenting and losing a few precious weeks in an endeavor to work some patent rotary pumps, which utterly failed, we came very near to a complete halt. There was no time to get proper steam-pumps of large dimensions constructed at any of the shops in Cincinnati, nor could we expect them in time from the East; every day's loss was irreparable—and so we were thrown back upon our own resources. Accordingly, I had four large square box-pumps constructed of 3-inch pine plank, strongly hooped, and 24 feet long; one pair 18 inches in the clear, the other 20 inches. Cast-iron gratings with large india-rubber flap-valves formed the piston and lower check-valves; heavy piston-rods, connected with chains which passed over sheaves, and were shackled to rods, extending over the coffer-dam down to the river. These pumps were put up vertically, in two pairs, one pair worked at a time. They were propelled by one of the engines of the then 'Champion,' No. 1, a powerful tow-boat, owned by Amos Shinkle, Esq., who generously placed it at my disposal. These pumps worked well and never fail; they threw mud and sand as effectively as pure water, and discharged 40 gallons at each lift.

"When the Cincinnati excavation was commenced, a strong oak sheet-piling was driven along the river-front to guard against the pressure of water. This, together with a solid embankment, proved a most efficient coffer-dam on the river side. Owing to its low stage, the river gave us no trouble at all. But by the great depth and extent of the foundation, most of the wells along the rising ground, back of Cincinnati, were laid dry. We drained their supplies, and had to pump them out; and this copious influx came from a quarter totally unexpected. The excavation, however, proceeded rapidly day and night, until all the clay and sand was removed, and a deep layer of coarse sand and gravel was laid bare. Soundings were now made by driving

long iron bars to the limestone shale underneath, which proved to be about 12 feet lower. A depth of over 12 feet below extreme low water was reached, and the question now arose, whether to go to the rock, to pile, or to lay down a solid timber platform.

“A compact bed of gravel, if left undisturbed, and protected against undermining and washing, stands next to a solid rock foundation, provided that unequal settling is guarded against. Had this tower been located inside of low-water mark, I should have decided upon going down to the rock, although one season’s loss would have been the consequence. Piling I considered inferior to the plan adopted, to say nothing of the loss of time. It was therefore decided to stop at the gravel, and to build a solid timber foundation up to low-water mark, thence to commence the masonry. If the timber could be obtained in time in sufficient quantity, the success of this kind of foundation was much more certain to be achieved, and with less risk and cost, than any other plan.

“The timber foundation, thus laid, forms a platform of 110 feet long by 75 feet wide, composed of twelve courses or layers next to the river, but stepped off towards the land side to eight courses, in consequence of the greater hardness of the gravel, almost equal to hard-pan. We were obliged to employ all kinds of timber, soft and hard, mixed, as white pine, oak, maple, hickory, button-wood, elm, beech. The length of logs also varied from 25 to 40 feet. They were all flattened and counter-hewed to an even thickness of 12 inches, leaving the other two sides rough. The courses were crossed at right angles; each stick was thoroughly secured by iron rag-bolts of 18 inches long and 1 inch in diameter. The vertical joints were left open and filled with clean gravel and broken stone. Every course was leveled off with the adze and then thoroughly grouted with cement grout before the next course was laid down. Care was also taken in breaking the longitudinal joints efficiently. A solid platform of timber, 110 feet long, 75 feet wide, 12 feet deep on the river side, and 8 feet on the land side, well put together in the manner

described, offers a foundation nearly as good as rock, provided it is guarded against undermining.

"The result has fully justified my expectation. I have not been able to discover any settlement during the progress of the masonry; its condition to-day proves the excellence of the foundation. There are 16,000 perches of 25 cubic feet, equal to 400,000 cubic feet, of solid masonry in each tower. Allowing 150 pounds as the average weight of one cubic foot, the total weight of one tower is 60 millions of pounds, or 30,000 tons net. The area of the timber foundation being $110 \times 75 = 8,250$ superficial feet, the weight upon each foot is 3.63 tons or 7,272 pounds, or $50\frac{1}{2}$ pounds per superficial inch. This is equivalent to a solid mass of iron of 15 feet depth. Now experience proves that such a weight of iron will be supported upon a clay floor, if its surface is well consolidated by tamping. In the case of high chimney-stacks, elevated 300 to 400 feet, a still greater pressure is sometimes produced upon each superficial foot."

The great Brooklyn Bridge, also constructed under John A. Roebling, had the towers landed on a few feet of sand overlying bed-rock, and the load was allowed to run up to $5\frac{1}{2}$ tons per square foot.

The bridges in London, England, are nearly all founded upon the stratum known as "London Clay," and the Charing Cross Bridge causes a pressure upon it of in the neighborhood of 9 tons per square foot, while on the Cannon street bridge the load runs to in the neighborhood of $6\frac{1}{2}$ tons per square foot. Both of these structures, however, have shown considerable settlement, and when the great Tower Bridge was designed it was decided to reduce these loadings very materially. Tests made by sinking a trial cylinder showed settlement under $6\frac{1}{2}$ tons per square foot, and, disregarding skin friction of the caisson and the buoyancy of the water, 4 tons was adopted as the safe load; although, taking these into account, the actual load per square foot was between 1 and 2 tons.

The cantilever bridge at Memphis, Tenn. (Fig. 113), constructed by George S. Morison, has piers founded upon pneu-

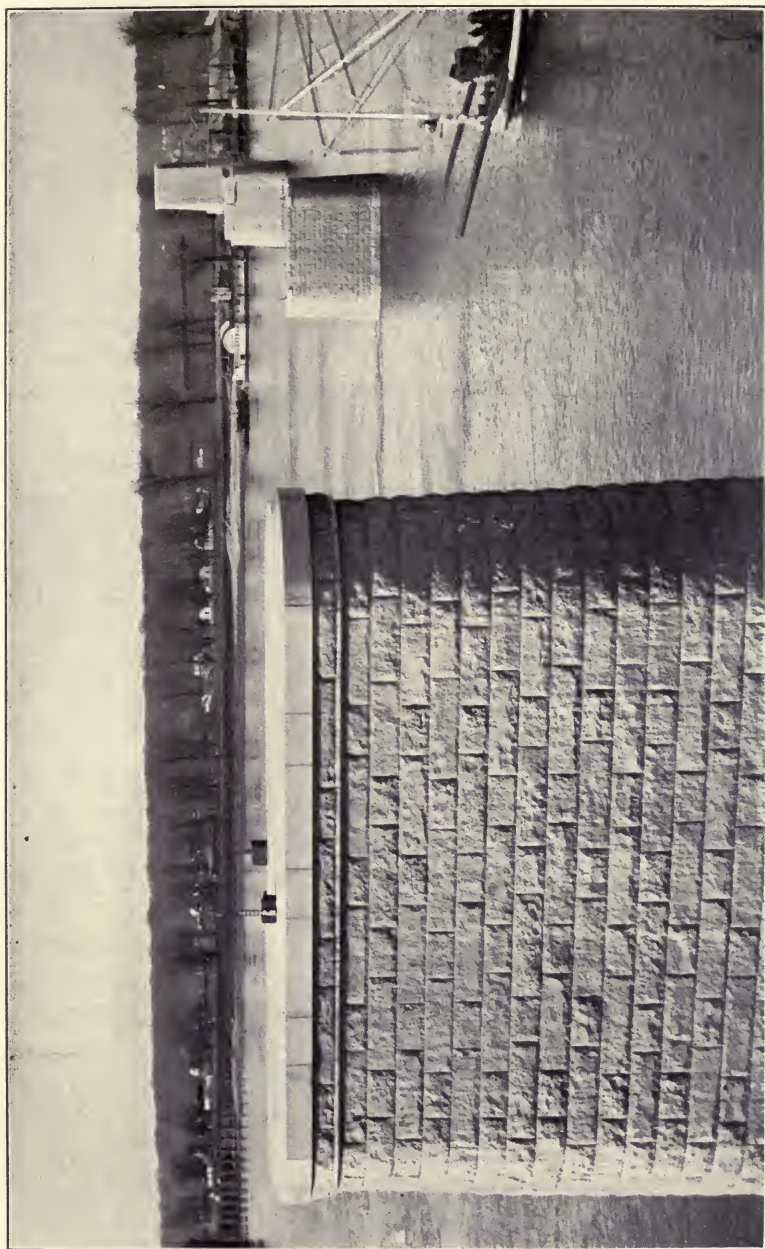


FIG. 113.—PIERS OF MEMPHIS CANTILEVER.

matic caissons, and tests were made to determine the maximum bearing capacity of the soil, which was a compact clay, and it was found to have an ultimate bearing capacity of $9\frac{1}{2}$ tons.

Although the foregoing represent what is the best practice in regard to the allowable loads per square foot on various classes of soil, they have in many cases been much exceeded. For example, the foundation of the Washington Monument causes a pressure on the very fine sand of 11 tons per square foot, although a maximum of about 14 tons is reached during a high wind. Similar overloaded conditions exist with many bridges, as the Gorai Bridge causes a pressure of 9 tons on the close-sand foundation, and nearly as much pressure is put upon the sand bottom in the Nantes Bridge, where pressure reaches $8\frac{1}{2}$ tons per square foot, although this has settled some, indicating that such high figures are nearer the ultimate bearing capacity than safe loads. A mixture of clay and sand on which the Szegeidin Bridge in Hungary is founded carries a load of $7\frac{1}{2}$ tons per square foot, although it was found necessary to relieve this foundation by driving piles.

The above data as to the carrying capacity of soil of various kinds may be supplemented by stating that the safe loads for soils per square foot may be rapidly increased for hard-pan, cemented gravel, and, of course, very largely increased for rocky ground, as in the case of the Roquefavour Aqueduct in France where the pressure reaches 15 tons per square foot.

Probably the most generally accepted values for foundation loads, that is, the amount which can be placed with safety upon a square foot of foundation bed, are those given by Prof. I. O. Baker in an article published in the *American Architect*, in which he states the minimum allowable load to be 25 tons per square foot for rock, of similar hardness as is used in the best ashlar masonry, 15 tons per square foot for rock equal to the best brick masonry, 5 tons per square foot for rock equal to poor brick masonry, 4 tons for dry clay, 2 tons for moderately dry clay, 1 ton for soft clay, 8 tons for cemented gravel and coarse sand, 4 tons for compact and well-cemented sand, 2 tons

for clean, dry sand, and 0.5 ton for quicksand and alluvial soils, although these figures can be increased from 25 to 100 per cent., depending upon circumstances and as the judgment of the engineer on the work may dictate.

The building laws of Greater New York are more generally used as a model and authority than any other building rules, and the following extracts cover the provisions as to foundations:

"Where no test of the sustaining power of the soil is made, different soils, excluding mud, at the bottom of the footings, shall be deemed to safely sustain the following loads to the superficial foot, namely:

Soft clay, 1 ton per square foot;

Ordinary clay and sand together, in layers, wet and springy, 2 tons per square foot;

Loam, clay, or fine sand, firm and dry, 3 tons per square foot;

Very firm, coarse sand, stiff gravel, or hard clay, 4 tons per square foot, or as otherwise determined by the Commissioner of Buildings having jurisdiction.

Where a test is made of the sustaining power of the soil the Commissioner of Buildings shall be notified so that he may be present in person or by representative. The record of the test shall be filed in the Department of Buildings.

When a doubt arises as to the safe sustaining power of the earth upon which a building is to be erected the Department of Buildings may order borings to be made, or direct the sustaining power of the soil to be tested by and at the expense of the owner of the proposed building.

The loads exerting pressure under the footings of foundations in buildings more than three stories in height are to be computed as follows:

For warehouses and factories they are to be the full dead load and the full live load established by this code, which also gives the loads from other buildings.

Footings shall be so designed that the loads will be as nearly

uniform as possible and not in excess of the safe bearing capacity of the soil, as hereinbefore given.

Every building, except buildings erected upon solid rock or buildings erected upon wharves and piers on the water-front, shall have foundations of brick, stone, iron, steel, or concrete laid not less than 4 feet below the surface of the earth, on the solid ground or level surface of work, or upon piles or ranging timbers when solid earth or rock is not found.

Piles intended to sustain a wall, pier, or post shall be spaced not more than 36 or less than 20 inches on centers, and they shall be driven to a solid bearing if practicable to do so, and the number of such piles shall be sufficient to support the superstructure proposed.

No pile shall be used of less dimensions than 5 inches at the small end and 10 inches at the butt for short piles, or piles 20 feet or less in length, and 20 inches at the butt for long piles, or piles more than 20 feet in length.

No pile shall be weighed with a load exceeding 40,000 pounds.

When a pile is not driven to refusal, its safe sustaining power shall be determined by the following formula: Twice the weight of the hammer in tons multiplied by the height of the fall in feet divided by least penetration of pile under the last blow in inches plus one. The Commissioner of Buildings shall be notified of the time when such test piles will be driven, that he may be present in person or by representative.

The tops of all piles shall be cut off below the lowest water-line.

When required, concrete shall be rammed down in the interspaces between the heads of the piles to a depth and thickness of not less than 12 inches and for 1 foot in width outside of the piles.

Where ranging and capping timbers are laid on piles for foundations, they shall be of hard wood not less than 6 inches thick and properly joined together, and their tops laid below the lowest water-line.

Where metal is incorporated in or forms part of a foundation it shall be thoroughly protected from rust by paint, asphaltum,

concrete, or by such materials and in such manner as may be approved by the Commissioner of Buildings.

When footings of iron or steel for columns are placed below the water-level, they shall be similarly coated, or inclosed in concrete, for preservation against rust.

When foundations are carried down through earth by piers of stone, brick, or concrete in caissons, the loads on same shall be not more than—

Fifteen tons to the square foot when carried down to rock;

Ten tons to the square foot when carried down to firm gravel or hard clay;

Eight tons to the square foot in open caissons or sheet-pile trenches when carried down to rock.

Wood piles may be used for the foundations under frame buildings built over the water or on salt-meadow land, in which case the piles may project above the water a sufficient height to raise the building above high tide, and the building may be placed directly thereon without other foundation.

Foundation walls shall be construed to include all walls and piers built below the curb level, or nearest tier of beams to the curb, to serve as supports for walls, piers, columns, girders, posts, or beams.

Foundation walls shall be built of stone, brick, Portland cement concrete, iron, or steel.

If built of rubble stone or Portland cement concrete, they shall be at least 8 inches thicker than the wall next above them to a depth of 12 feet below the curb level; and for every additional 10 feet, or part thereof, deeper they shall be increased 4 inches in thickness.

If built of brick, they shall be at least 4 inches thicker than the wall next above them to a depth of 12 feet below the curb level; and for every additional 10 feet, or part thereof, deeper they shall be increased 4 inches in thickness.

The footing or base course shall be of stone or concrete, or both, or of concrete and stepped-up brickwork, of sufficient thickness and area to safely bear the weight to be imposed thereon.

If the footing or base course be of concrete, the concrete shall not be less than 12 inches thick.

If of stone, the stones shall not be less than 2' $\times$ 3', and at least 8 inches in thickness for walls; and not less than 10 inches in thickness if under piers, columns, or posts.

The footing or base course, whether formed of concrete or stone, shall be at least 12 inches wider than the bottom width of walls, and at least 12 inches wider on all sides than the bottom width of said piers, columns, or posts.

If the superimposed load is such as to cause undue transverse strain on a footing projecting 12 inches, the thickness of such footing is to be increased so as to carry the load with safety.

For small structures and for small piers sustaining light loads, the Commissioner of Buildings having jurisdiction may, in his discretion, allow a reduction in the thickness and projection for footings or base courses herein specified.

All base stones shall be well bedded and laid crosswise, edge to edge.

If stepped-up footings of brick are used, in place of stone, above the concrete, the offsets, if laid in single courses, shall each not exceed 1½ inches, or if laid in double courses, then each shall not exceed 3 inches, offsetting the first course of brickwork, back one-half the thickness of the concrete base, so as to properly distribute the load to be imposed thereon.

If, in place of a continuous foundation wall, isolated piers are to be built to support the superstructure, where the nature of the ground and the character of the building make it necessary, in the opinion of the Commissioner of Buildings having jurisdiction, inverted arches resting on a proper bed of concrete, both designed to transmit with safety the superimposed loads, shall be turned between the piers. The thrust of the outer piers shall be taken up by suitable wrought-iron or steel rods and plates.

Grillage beams of wrought iron or steel resting on a proper concrete bed may be used. Such beams must be provided with separators and bolts inclosed and filled solid between with con-

crete, and of such sizes and so arranged as to transmit with safety the superimposed loads.

All stone walls 24 inches or less in thickness shall have at least one header extending through the wall in every 3 feet in height from the bottom of the wall, and in every 3 feet in length, and if over 24 inches in thickness, shall have one header for every 6 superficial feet on both sides of the wall, laid on top of each other to bond together, and running into the wall at least 2 feet.

All headers shall be at least 12 inches in width and 8 inches in thickness and consist of good flat stones.

No stone shall be laid in such walls in any other position than on its natural bed.

No stone shall be used that does not bond or extend into the wall at least 6 inches.

Stones shall be firmly bedded in cement mortar and all spaces and joints thoroughly filled."

CHAPTER XII.

LOCATION AND DESIGN OF PIERS.

PIERS of a bridge cannot always be located with reference to easy construction nor spaced at economical distances apart. In thickly settled parts of a country, or as part of an existing line of communication, the bridge must be located usually in a position previously determined, and the piers can only be spaced with regard to economy, provided due regard can at the same time be paid to the needs of navigation, government requirements, and sufficient waterway.

Where the bridge is to be constructed in a new country, or upon a new line of road, the crossing should be selected where the river is of moderate width; that is, not so wide as to demand a structure of excessive length and probably of excessive cost, nor so narrow that the current will be exceedingly swift and make the foundations very difficult and costly to build, unless, of course, it is narrow enough to admit of using a one-span structure at a reasonable cost.

On all the large navigable rivers, the channel is fixed and the length of the channel span prescribed by law, as is also the method of procedure in obtaining the approval of the government engineers. The Secretary of War must be furnished with a copy of the State law authorizing the construction of the bridge, certified to by the Secretary of State under seal; drawings in triplicate showing the general plan of the bridge; a map in triplicate showing the location of the bridge, giving, for the distance of one mile above and one-half mile below the proposed location, the high- and low-water lines upon the banks of the stream, the direction and strength of the

current at high and low water, with the soundings accurately showing the bed of the stream, and the location of any other bridge or bridges, such map to be sufficiently in detail to enable the Secretary of War to judge of the proper location of the bridge. In addition to the above, if the applicant is a corporation, there will be required a certified copy of its articles of incorporation, a certified copy of the minutes of the organization of the company, and an abstract of the minutes of the corporation, showing the present officers of the company, all duly certified to.

When the location of the bridge has been made, a thorough examination of the site must be instituted. Soundings must be made to determine the depth of the stream at low water; ordinary and extreme high-water lines must be established and the flow of the stream be obtained. A careful examination must be carried out as to the character of the river bed, and drillings made to learn the character and thickness of strata and the distance to bed-rock, as well as the quality of it.

Borings to a small depth may be made by hand-drills (Fig. 114, *A*), which are operated by striking with a sledge and turned constantly

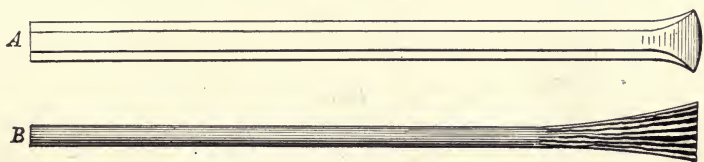


FIG. 114.—HAND-DRILL AND SWAB.

to keep a round bore, or if long and heavy they will cut their way, if simply raised up and allowed to drop, with their own weight. The hole is kept partly filled with water and can be cleaned out with a small sand-pump or with a swab (Fig. 114, *B*) made from a stick slivered at the end, which will also bring up samples.

The Pierce steel prospecting auger is a tool which can also be used without a derrick to bore test holes from 10 to 50 feet into loose soils or clay. Holes from $2\frac{1}{2}$ to 6 inches in diameter can be drilled and samples obtained. The auger can be turned either by hand-wrenches or by horse-power.

Where the borings are to be of an extensive character a well-drilling machine can be utilized, such as shown in Fig. 115, and

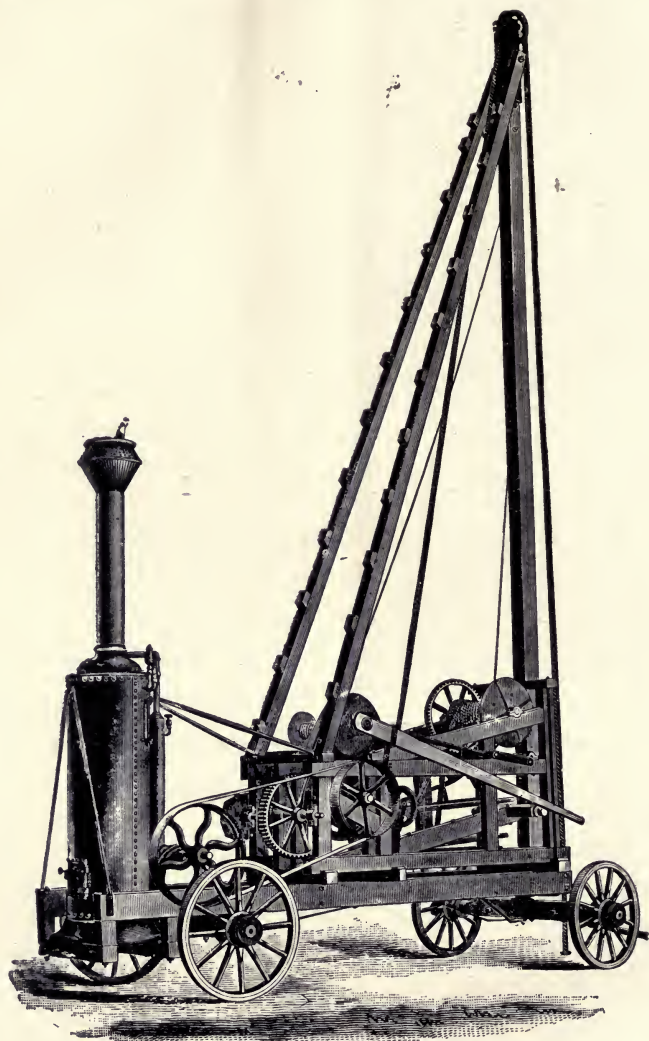


FIG. 115.—STEAM-POWER WELL-DRILLER.

which can be run onto an ordinary flatboat and towed to place.

The tools for drilling are a temper screw for regulating the

height of the drill, a sinker bar to give the weight, steel jars, and drilling-bits. A sand-pump is used to clean the hole and obtain samples; rope-spears, rope-knives, and fishing-tools to remove lost rope, tools, and pebbles or other obstructions. The drill holes, unless through rock, are cased with iron pipe which can be withdrawn when the hole is completed.

The borings made by the Mississippi River Commission were very extensive and a special tripod apparatus (Fig. 116) was devised with a view to easy transportation and easy repair in the field. The tripod was 30 feet in height, with a strong head or cap, surmounted by a galvanized-iron guide-pipe 20 feet in height, in two sections, and held in place with guy-ropes. The men operating the tools stood upon the triangular platforms which were attached to the legs. The casing was iron pipe in 10-foot lengths and screwed together so as to present a smooth surface, while the bottom was provided with a steel cutting-shoe, having a mouth slightly larger than the pipe. The sinking is accomplished by driving and twisting, the driving being done by means of the clamp on the pipe and the maul sliding on the pipe. (Fig. 117.) The weight of the maul is from 80 to 100 pounds and is worked by three men giving it a lift of about 2 feet, the best results being obtained when the men act in concert and give rapid blows. The removal of the core and samples is accomplished by means of the various tools shown in Fig. 116, and requires great care and considerable experience. The pump was raised and lowered by means of the reel attached to one leg of the tripod, and its distance from the surface noted from graduations on the pump-rod. When the boring is completed the tube is withdrawn by a system of compound levers, assisted by a set of differential blocks when necessary, as the force exerted was often as much as the strength of the pipe at the joints. The pebble-tongs were for use in removing large pebbles which would not enter the pumps, and for recovering lost tools or the pump itself in case of becoming detached.

The above account is taken from the report of J. W. Nier, assistant engineer, to which reference must be made for other details.

When the examination of the site has been completed and the borings finished, the form of foundations may be decided

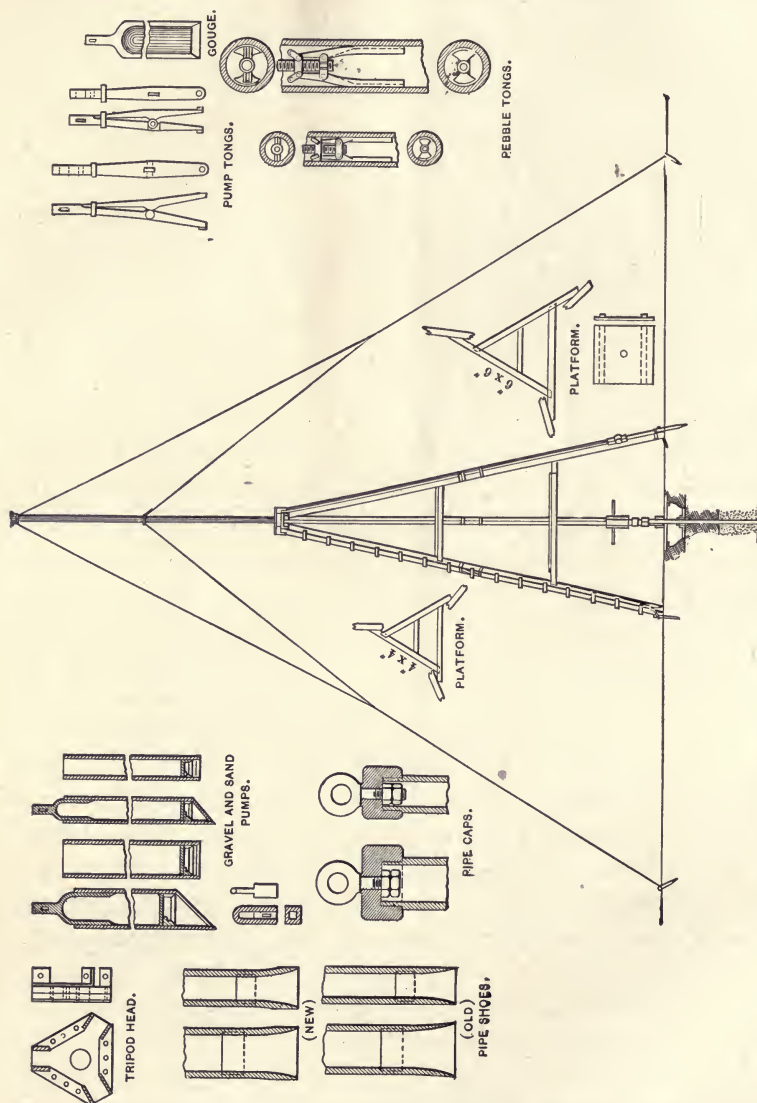


FIG. 116.—TEST-BORING APPARATUS, MISSISSIPPI RIVER COMMISSION.

upon, due weight being given to good foundations and to the allowable expenditure. Should the obtaining of good founda-

tions be seen to be very expensive, long spans must be adopted to require few piers in the river; but if inexpensive, much shorter spans, with more piers, may be used.

The length of spans for a least cost of structure was formerly assumed to be decided when the cost of one span was made equal to the cost of one pier, and for spans of certain capacity this might be approximately true, but a very neat mathematical solution of this problem by Alfred D. Ottewell, consulting engineer, was published in the *Engineering News* of December 14, 1889. The total length of the structure in feet was represented by l , the number of spans by n , the length of one span in feet $l \div n$ by s , the cost of one span in dollars by c , the cost of one pier in dollars by p , the total cost of the structure in dollars by y , while a and b are constants.

From the estimated cost of a large number of spans, a curve of costs was plotted and the following equation of a parabola deduced:

$$c = a + \frac{(s - 20)^2}{b} \quad \dots \quad (1)$$

Since there are n spans and $n + 1$ piers, the total cost of the structure would be

$$y = nc + (n + 1)p \quad \dots \quad (2)$$

Then by substituting the value of c from (1), reducing and making the first differential coefficient equal to zero, the cost of one pier is obtained, which will make the total cost of the structure a minimum, or

$$p = \frac{s^2 - (ab + 400)}{b} \quad \dots \quad (3)$$

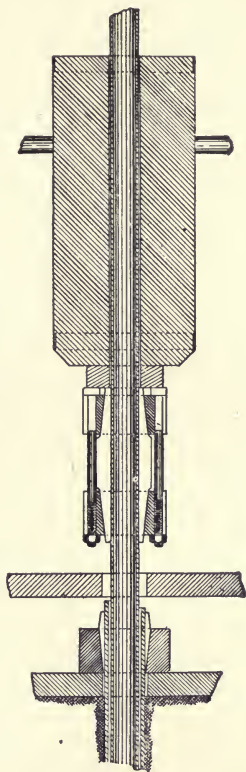


FIG. 117.—CLAMP AND MAUL.

Or when the cost of a pier has been estimated, the economical length of span may be found by a transposition of the above formula:

$$s = \sqrt{ab + 400 + pb}. \quad . \quad . \quad . \quad . \quad . \quad (4)$$

The values of a and b may be found by substituting in equation (1) computed values of the cost of a number of spans for an actual loading. Values of s , p , and c may then be computed and tabulated for spans from 100 feet upwards, as formula (1) is not true for shorter lengths.

In an actual calculation for B. & O. R.R. loading, which consists of two 125-ton engines followed by a 4,000-pound per lineal foot train-load, a was found equal to 1,950 and b to 3.05. Assuming a case where the length of the bridge is 700 feet, where the height of the piers will average 25 feet, and the average cost of piers and abutments be \$4,310, then from formula (4) the economical span will be found equal to 160 feet. The total cost of the structure will be found, by using formula (1), and the cost of piers as above, to be \$59,700; while with only four spans of 175 feet the total cost would exceed \$60,800, and with six spans of 117 feet would be about \$61,400.

Should there be any doubt as to the ease of obtaining foundations, the prudent engineer might deem it wise, however, to build the four-span structure and avoid the risk and delay which would be caused by another foundation in the river.

After deciding upon the number and location of the piers, they must be designed with reference both to their being as slight obstructions to the water as possible and to their architectural appearance.

The design of piers has been given particular attention by the late Geo. S. Morison, consulting engineer, whose work on the bridges across our great rivers is notable for its strength, simplicity, and finished appearance. In a lecture he described the process of the design of some large piers: "Fourteen years ago I had occasion to design a bridge pier for a bridge across one of our Western rivers, and I tried to make an ornamental pier. When

the plans were completed I did not like them. One change after another was made, all tending to simplicity. Finally the plans were done. From high water down, the pier was adapted to pass the water with the least disturbance; it had parallel sides and the ends were formed of two circular arcs meeting. Above high water the ends were made semicircular instead of being pointed. The pier was built throughout with a batter of one in twenty-four. A coping 2 feet wider than the body of the pier projected far enough to shed water, and the projection was divided between the coping and the course below. Another coping with a less projection surmounted the pointed ends where the shape was changed. It was as simple a pier as could be built, and in every way fitted to do its duty. I had started to make a handsome pier. The pier that was exactly what was wanted for the work was the only one that satisfied the demands of beauty. Forty-three piers of precisely this design [no change having been made except in the varying dimensions required for different structures], besides eight others in which only the lower parts are modified, are now standing in eleven different bridges across three great Western rivers. In designing a pier it must be remembered that the portion of the pier below the water has more to do with the free passage of the water than that above water. In a deep river the model form of the pier should begin near the bottom of the river and not at low water. Many rivers in flood time carry a great amount of drift. A pier like that which I have described catches but little of this drift. If, however, a rectangular foundation terminates but little below water, that foundation may both disturb the current and catch the drift."

The piers of the Omaha Bridge which carries the Union Pacific across the Missouri River are illustrated in Fig. 118, and were constructed as described and are among the most beautiful piers in this country.

In Europe, where money is more lavishly expended on great works of engineering, piers of great architectural beauty are much more frequent. The Russian Government railways,

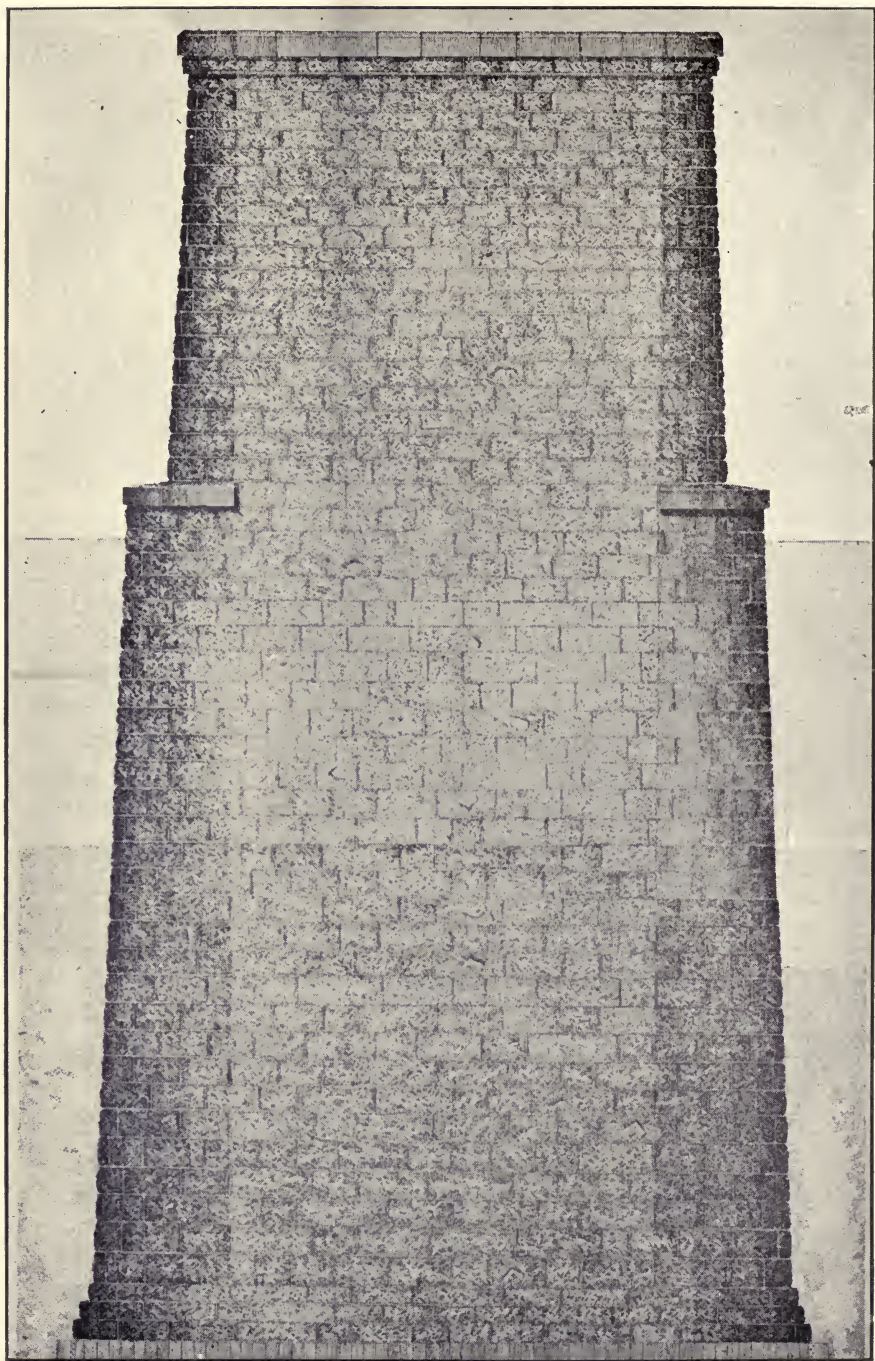


FIG. 118.—PIER OF OMAHA BRIDGE, UNION PACIFIC SYSTEM.

which have seemingly been constructed without regard to expense, have many beautiful examples of bridge masonry and piers; the view of one of them (Fig. 119), with curved ends, shows the elegant and massive character of the masonry. While extremely

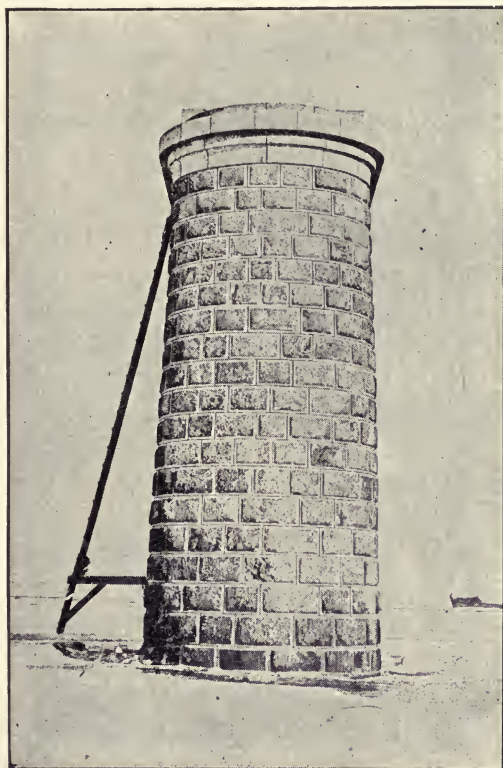


FIG. 119.—RUSSIAN PIER, RUSSIAN STATE RAILWAYS.

simple in design, the cut-stone coping and the molded corbel course below give it a finish which cannot be surpassed.

The design of piers for strength and stability is fully treated in Baker's "Masonry Construction," but some experiments, which were made with reference to the proper form to occasion the least resistance, will be quoted at length from Cresy.

The introduction of piers into a channel gives rise to a great disturbance in the velocity and flow of the water. Rapid cur-

rents are formed which cause the bed of the stream to become washed and the foundations to be endangered; eddies are created which are likewise undesirable, and it becomes necessary to adopt such a form for the ends of the piers that the disturbance to the flow shall be small.

M. Bossut, in a French work on jetties, thought to have solved this problem by mathematics, his conclusion being that the starling should be triangular, the nose being a right angle.

M. Dubuat, in his "Principles of Hydraulics," gave another solution which was more nearly the truth, in that he arrived at the conclusion that the faces of the starling should be convex curves. The true form is most nearly reached when these curves are tangent to the sides of the pier, and, further than this, regard must be paid to giving enough solidity to the starlings to protect them from ice and drift. A happy medium would seem to be reached by making the curves with a radius equal to one-sixth of the circumference, described on the sides of an equilateral triangle.

Experiments were made with models of different forms, which were placed in a rectangular canal between boards of 50 centimeters in length, in which the water flowed about 40 millimeters in height, the models being 15 centimeters in thickness. By means of a fall, the water was given a velocity of 3 meters 9 centimeters per second, the contraction, eddies, and currents being carefully measured. The first experiment was made on a pier (Fig. 120, *a*) with rectangular starling. An eddy was formed before the pier 34 millimeters high, in a nearly circular band A, falling nearly vertical at the corner. There were two other currents along the faces of the pier, the height of which can be seen in the cross-sections.

The second experiment (Fig. 120, *b*) was with a triangular starling, the nose being a right angle. It formed a less obstruction than the square end, but the fall at the shoulder was as deep and more dangerous, while eddies were formed as seen in the sections.

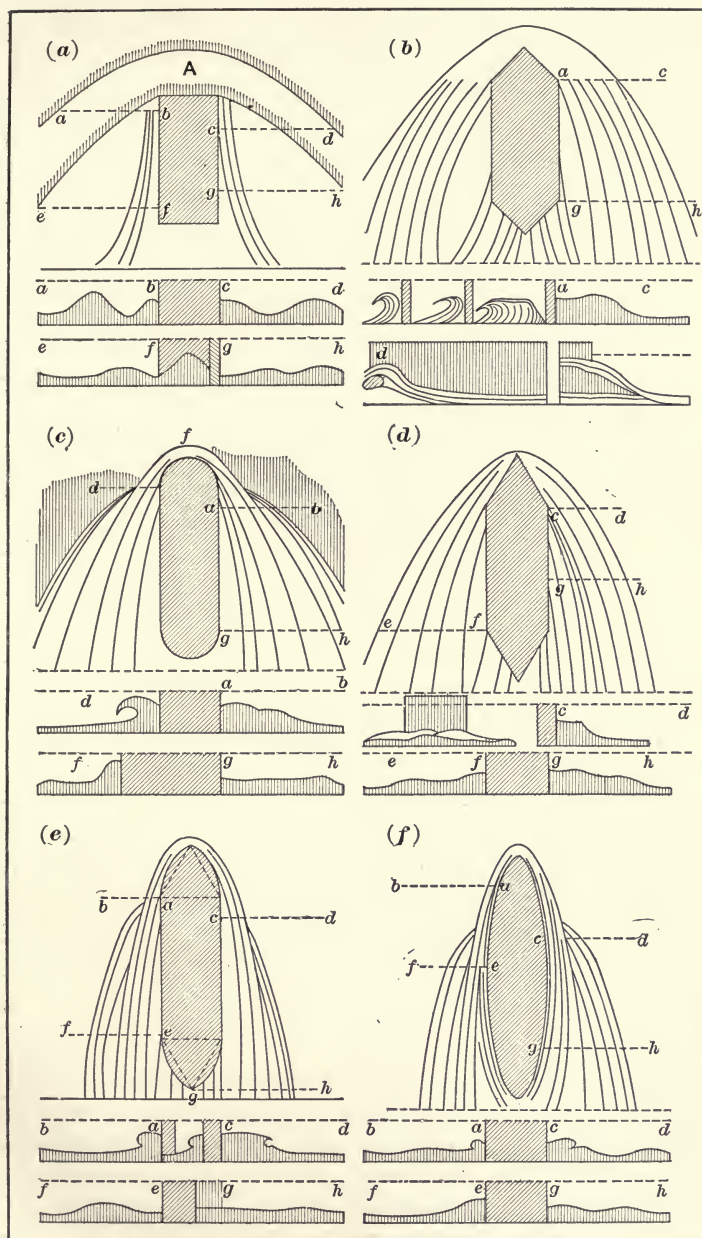


FIG. 120.—CRESY'S EXPERIMENTS ON THE FORM OF PIERS.

The third one (Fig. 120, *c*) had a semicircular starling. The eddy was not so wide, but nearly as high.

The fourth model had a triangular starling, with an angle of 60° at the nose (Fig. 120, *d*). The eddy was less, as was also the fall at the shoulder.

The starling in the fifth was formed by two circular arcs, tangent to the sides and described on the sides of an equilateral triangle (Fig. 120, *e*). The eddy was small and there was no fall at the shoulder.

The sixth (Fig. 120, *f*) was a model the plan of which was an ellipse, of which the small diameter was one-fourth the length and the eddy was less than any of the others.

The seventh model (Fig. 121, *a*) had a starling with concave faces, such as is sometimes used where the wing-wall meets an abutment. It produced the most dangerous currents of all.

The eighth (Fig. 121, *b*) was of the same form as Fig. 120, *e*, but the water was supposed to mount the springing of the arch.

The ninth and tenth experiments (Figs. 121, *c* and 121, *d*) were on the same forms as Figs. 120, *e* and 120, *f*, but the current had a velocity of 4 meters 87 centimeters per second, such as a river would have in its overflow. The eddy (Fig. 121, *c*) rose to nearly twice the height, as was the case with the lesser velocity, and, while there was no fall, the inclination formed along the faces was more rapid.

The effect with this velocity on the elliptical pier (Fig. 121, *d*) was similar to the lesser velocity, but more marked. It may thus be concluded that the elliptical section offers the least resistance to the current and occasions the least contraction, while the form with convex starling comes next, and of piers with triangular starlings the one with the 60° nose is the best.

Where ice is to be provided for, the nose is often inclined to allow large cakes to mount it and break in two, without doing further damage. For any large or important structure, the design of the piers should receive a great deal of study, and be designed not only with reference to their theoretical form, but with reference to the form of pier which has shown the best

results practically and has been found to be best suited to the velocity of the stream in which they are to be built, and to best

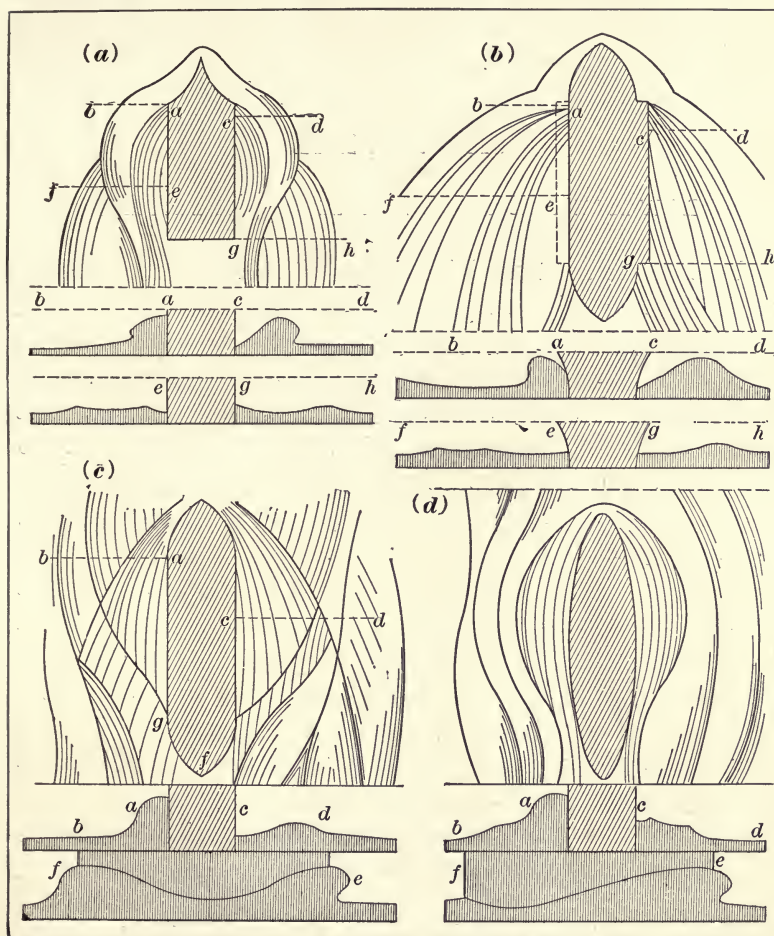


FIG. 121.—CRESY'S EXPERIMENTS ON THE FORM OF PIERS.

withstand the drift and ice that may be met with, giving at the same time all the consideration possible to the architectural effect and to the harmony with the entire structure.

CHAPTER XIII.

LOCATION AND DESIGN OF PIERS (CONTINUED).

THE stones most used in the building of piers are granite, sandstone, marble, and limestone; the amount of granite used for this purpose being about the same or possibly a little less than sandstone. Next to these, limestone is used most largely and marble the least of the four, owing to the fact that most marble is suitable for dressed stonework for buildings and too expensive for use in piers.

Granite is most largely produced in the New England States, and notably in Maine, Massachusetts, and Vermont. Next to these States comes California with about as much of an output as Vermont. Only fifteen of the States in the Union do not have an output of granite, so that it may be said to be available at greater or less cost in any part of the United States.

Sandstone is most largely produced in Pennsylvania, Ohio, and New York, although but thirteen of the States are non-producers of this most commonly used building material.

Marble is most largely produced in Vermont, although New York, Tennessee, and Georgia have extensive quarries and are large producers.

Limestone is extensively quarried in Pennsylvania, Ohio, New York, Missouri, Wisconsin, Illinois and Indiana, only six States being without output of this stone.

Granite is the best building stone for use in constructing piers, and under this head are included the true granites, gneiss, mica-schist, andesite, syenite, and quartz-porphry.

Sandstone covers all the consolidated sands, the strength of



FIG. 122.—GENERAL VIEW OHIO FREESTONE QUARRY.

sandstone depending entirely upon the cementing material, as it is simply quartz grains cemented together, and when silica is the cementing material it is the very best. Quarrymen term sandstone according to its quality as bluestone, freestone, and conglomerates.

Limestone is usually the least valuable of stone that is used in building piers, owing to the fact that it is too soft and stands the weather poorly. It consists practically of amorphous calcium carbonate, sometimes cemented together by crystalline substances with many impurities. Under the head of limestone is also usually included magnesium carbonate, called magnesium limestone, and when the stone is about equally made up of carbonate of lime and magnesium it is termed dolomite. Dolomite is much harder than ordinary limestone, and consequently forms a much better building material. Chemically, there is little difference between limestone and marble, except that marble has been crystallized by the action of heat.

Of the quality of the granite to be obtained in any part of the United States very little need be said, as it is always a desirable building material, and only the question of cost comes up for consideration as to whether it can be used or not. In the New England States, Georgia, California, and Washington it is cheap enough to make it possible to use it wherever a reasonable amount of money is available for the construction of bridge piers or foundations of any sort. The softer sandstones obtainable throughout the country and which have a compressive strength of about 5,000 pounds per square inch are suitable for building bridge piers where not too much ice is to be contended with or abrasive action of any character.

The freestones of northeastern Ohio, and similar stone wherever found throughout the United States and similar to the Chuckanut stone of Puget Sound, are almost as desirable as granite for the construction of piers.

Limestone does not very often occur of such quality as to warrant its use in masonry construction, and while it may seem to be of sufficient hardness to warrant consideration, attention need

only be called to its use in the State Capitol building of Ohio, where it has weathered badly, to make it seem advisable to find some other material if possible.

Where marble is plenty, it is of course of sufficient strength and hardness to make it desirable for use in foundation work and for piers, provided the cost is not excessive, and a poorer quality of marble found in Tennessee and Georgia, and known as iron limestone, is certainly one of the best building stones to be found anywhere. The most famous limestone to be found in the United States is the oolitic limestone of Indiana. It is very easily quarried and hardens on exposure to the air, so that it is quite durable; and, on account of the large size of the blocks in which it can be gotten out, is very much used for massive work.

One of the things least often considered in the selection of stone for ordinary piers is the color; although, in the case of piers for city bridges, towers for suspension bridges, or work of this character it is very desirable to have the material of pleasing appearance, and this is most readily found in the granites and marbles. The ordinary gray or bluish gray of many of the sandstones is also very pleasing, and in many cases other colors can be found for belt courses, copings, and trimmings of various kinds.

While the color of stone is apt to change considerably after it is quarried, it is usually possible to know what the change will amount to, by seeing stone of the same kind that has been in use. The gray color of many of the granites is due to a mixture of light feldspar with a dark-colored mica and very fine hornblende. The sedimentary rocks are colored with iron and various other minerals, and it is always necessary when iron is the coloring-matter to make sure that in weathering the stone will not turn rusty.

One of the most important qualities to be taken into account in selecting building stone is its durability under changes of temperature and abrasion, although the stone may prove valueless owing to chemical changes due either to its going to pieces by the action of the water, carbon dioxide, or some of the organic acids. The change of temperature affects rock by the unequal

expansion and contraction of the various minerals composing it, or the rock may be so porous as to become filled with water, and when this freezes the expansion of the ice will cause it to crack or break. The report on the building stones of Wisconsin states that "the expansive force of heat is well shown in many of the limestone quarries of Wisconsin, where beds from 5 to 6 inches in thickness are for the first time exposed to the heat of the summer sun. These thin beds become heated throughout their entire thickness, and arch up on the floor of the quarry, generally breaking and completely destroying the stone." The effect of freezing and thawing is well stated in Volume II of the Washington Geological Survey as follows: "All rocks are more or less porous; and these pores, before the rock is quarried, always contain more or less water, and after being quarried for some time and exposed to the atmosphere they lose this water. However, when rain-storms occur they are apt to absorb more water, and if the temperature falls below the freezing-point when the stone is in this condition the water will be frozen. As is well known, water on freezing expands and in expanding exerts a pressure or expansive force equal to about 150 tons to the square foot. It is plain to see that if the rock contains any very large amount of water and freezes, the result will be the spreading apart or separating somewhat of the particles composing the stone. Then the water thaws and freezes again, and so on indefinitely, and the result is that particles are finally completely loosened and fall out. This effect is principally on the surface of the rock and is the cause of the scaling frequently seen in buildings that are built of certain kinds of stone. In addition to the pores, which occur in the rocks, there are the openings which occur along the joint, bedding and foliation planes. Water falling on the surface of the ground and more or less of it sinking into it enter these cracks or crevices; and while it will flow more readily along these than it does through the pores of the rock, still many times these will be filled and freeze while in that condition. As has already been stated, water when freezing exerts a very great expansive force and will tend to separate the rocks

along these planes, and when the ice melts the rocks do not come back to their original position, but retain the position they had when the water was frozen in them. Then these cracks are filled again and refrozen, and the seams opened a little farther, and the same process repeated time after time finally produces a perceptible effect and tends to weaken the stone. This is especially true of sedimentary deposits such as sandstones, particularly where they have marked bedding planes."

The effect of ice, driftwood, and the like is to abrade the surfaces of piers so that with stone that is at all soft it becomes a very serious matter. The action of sand carried by wind-storms, while very destructive, can hardly be considered in the design of piers, as it is seldom that they are so situated as to be affected in this way.

The mineralogical composition of stones has a very important bearing upon their durability, but, as this is fully treated in works on mineralogy, it will not be gone into here.

Chemical and microscopic examinations are often of value, and for any large piece of work, or where the stone as proposed for use has not been used for any great length of time, these tests should be made; but, as a general rule, the physical tests are of very much greater value. Nearly all of our universities and many of the larger engineering offices now have testing-machines (Fig. 123), so that tests can easily be made. They should be carried out by standard methods, so that they will be of value for tabulation with the results of other investigators, and of use to future consumers of the stone.

It was formerly the custom to make tests on 1-inch cubes, but wherever the testing-machine is large enough they should be not less than 2-inch cubes, having 4 square inches of area, or larger if possible, as stone in large pieces has greater resistance per square inch, and thus the actual strength of the stone will be more nearly determined.

The Wisconsin report divides the physical tests into two divisions: First, strength tests, comprising crushing strength, transverse strength—giving the modulus of rupture, and the coefficient of elasticity; second, durability tests, covering specific gravity,

porosity, weight of the stone per cubic foot, effect of extreme heat, effect of alternate freezing and thawing, action of carbonic-acid gas, and the action of sulphurous-acid fumes.

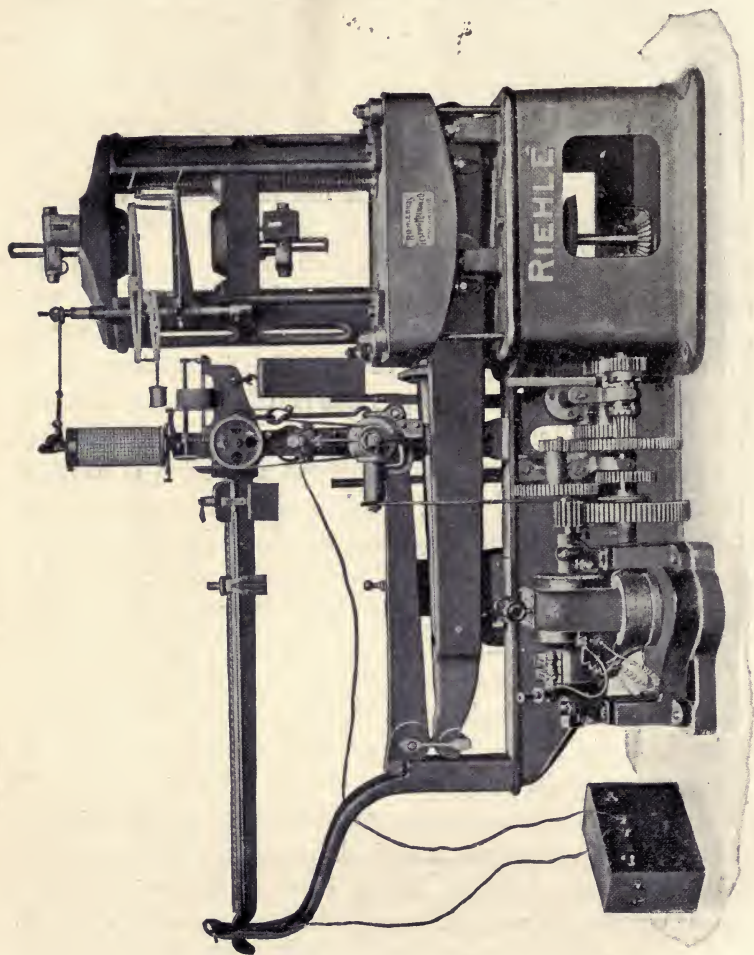


FIG. 123.—TESTING-MACHINE.

The crushing strength has usually been considered all that is necessary, but the above report speaks of this test as follows: "It has been computed that the stone at the base of the Washington Monument, the highest structure in the world, sustains a maximum pressure of 22,658 tons per square foot, or 314.6 pounds

per square inch. Certain contractors require a stone to withstand twenty times the pressure to which it will be subjected in the wall, while others only require ten times that pressure. Even if requiring a factor of safety of twenty, the strength required for a stone at the base of this monument would be only 6,292 pounds per square inch. The pressure at the base of our tallest building can scarcely exceed one-half that at the base of the monument, or 157.3 pounds per square inch. According to the above estimate, stone used in the tallest buildings does not require a compressive strength above 3,146 pounds per square inch. There is scarcely a building stone of importance in the country that does not give a higher test than this. Ordinary building stone has from two to ten times the maximum required crushing strength. A stone having a crushing strength of 5,000 pounds per square inch is sufficiently strong for any ordinary building." So that it will be seen that very few stones will not be strong enough in this regard.

The following tables, which are taken from the Washington Geological Survey, give a large number of tests which have been made on building stone in various parts of the country:

TABLE II.—CRUSHING STRENGTH IN POUNDS PER SQUARE INCH, SPECIFIC GRAVITY, AND RATIO OF ABSORPTION OF BUILDING STONE.

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
GRANITE.			
(1)*Montello, Wisconsin.....	43,973	2.639	.079
(1) Granite City, Wisconsin.....	25,000	2.675	.133
(1) Berlin, Wisconsin.....	32,747	2.643	.143
(1) Granite Heights, Wisconsin.....	16,723	2.631	.180
(2) East St. Cloud, Minnesota.....	28,000	2.692	2.59*
(2) Sauk Rapids.....	21,500	2.710	.190
(2) Beaver Bay, Minnesota.....	20,750	2.69	.140
(3) Fourche Mountain, Arkansas.....	20,000	2.642	1-1673
(3) Fourche Mountain, Arkansas.....	28,700	2.635	
(3) Fourche Mountain, Arkansas.....	21,500		
(4) Little Rock, Arkansas.....	22,388		
(4) Little Rock, Arkansas.....	17,407		
(4) Milbridge, Maine.....	19,917		

* See p. 2.6 for references.

TABLE II (CONTINUED).

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
SANDSTONE.			
(2)*Hinckley, Minnesota.	19,000	2.470	4.88
(2) Dresbach, Minnesota.	6,500	2.380	11.48
(2) Jordan, Minnesota.	4,750	2.340	12.69
(1) Ablemans, Wisconsin.	13,669		
(1) Albemans, Wisconsin.	11,030		
(1) Ablemans, Wisconsin.	8,602		
(1) Ablemans, Wisconsin.	10,056		
(1) Dunnville, Wisconsin.	2,502	2.601	15.130
(1) Port Wing, Wisconsin.	5,498	2.638	10.330
(1) Houghton, Wisconsin.	4,549		
(1) La Valle, Wisconsin.	13,350		
(1) Bayfield, Wisconsin.	4,588	2.639	4.760
(5) Birdsboro, Pennsylvania.	11,448		
(5) Waltonville, Pennsylvania.	14,000	2.350	1-27
(5) Waltonville, Pennsylvania.	12,730		
(5) Lumberville, Pennsylvania.	22,250	2.660	
(5) Laurel Run, Pennsylvania.	17,600	2.660	1-900
(5) White Haven, Pennsylvania.	29,252		
(5) Portland, Connecticut.	12,580	2.350	1-40
(5) Middletown, Connecticut.	6,250	2.360	1-40
(5) E. Longmeadow, Massachusetts.	12,330	2.480	
(5) Medina, New York.	16,031	2.400	1-53
(5) Marquette, Michigan.	6,150		
(6) St. Anthony, Indiana.	3,000		3/40
(6) Riverside, Indiana.	6,100		1/25
(6) Riverside, Indiana.	6,800		1/50
(6) Worthy, Indiana.	6,825		
(6) Berea, Ohio.	11,213	2.110	1/20
(6) Hummelstown, Pennsylvania.	12,810		
(6) Gunnison, Colorado.	5,250	2.200	.09
(6) Cleveland, Ohio.	6,800	2.240	1/37
(6) N. Amherst, Ohio.	5,450	2.140	1/69
(6) Angel Island, California.	4,574	2.730	1/31
(6) San José, California.	2,400	2.640	1/16
(6) Bass Island, Wisconsin.	4,850		

* See p. 206 for references.

The engineer should always make careful inquiries as to whether the manner of quarrying stone injures it in any way, as the use of explosives will very materially shatter many kinds of stone and practically ruin them for heavy construction work.

Most all quarrying (Fig. 122) is now done by the use of drills or channeling-machines, so that damage from explosives is not so frequently found as formerly.

The larger granite quarries use no machinery in quarrying the stone other than rock drills and hoisting-apparatus. Lewis holes are drilled close together in groups of two or three, the partitions broken down, and when a series of these have been drilled around the piece to be blasted out, explosives are used to loosen the rock, and by this method the action is wedge-like, and the rock is not damaged.

In marble-quarrying channeling-machines are used, which are moved back and forth on narrow tracks and cut vertical channels 5 or 6 feet or over in depth, and a little over an inch in

TABLE III.

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
LIMESTONE.			
(1)*Knowles, Wisconsin.	29,189	2.793	1.76
(1) Bridgeport, Wisconsin.	10,112	2.740	5.49
(1) Bridgeport, Wisconsin.	6,675	2.740	5.46
(1) Duck Creek, Wisconsin.	23,783	2.843	.419
(1) Sturgeon Bay, Wisconsin.	31,957	2.841	.19
(1) Sturgeon Bay, Wisconsin.	39,983	2.700	.64
(1) Genesee, Wisconsin.	36,731	2.833	1.10
(1) Genesee, Wisconsin.	29,253	2.829	1.15
(1) Marblehead, Wisconsin.	42,787	2.856	.31
(1) Lannon, Wisconsin.	31,936	2.814	1.32
(1) Fountain City, Wisconsin.	8,830	2.804	4.95
(1) Wauwatosa, Wisconsin.	19,111	2.821	2.29
(1) Wauwatosa, Wisconsin.	13,406	2.826	2.53
(1) Wauwatosa, Wisconsin.	23,744		
(2) Red Wing, Minnesota.	23,000	2.750	2.95
(2) Stillwater, Minnesota.	10,750	2.590	2.19
(1) Kasota, Minnesota.	18,500	2.640	2.51
(2) Mantorville, Minnesota.	9,500	2.650	5.37
(2) Minneapolis, Minnesota.	21,750	2.770	2.36
(7) Ellettsville, Indiana.	6,700		1-31
(7) Ellettsville, Indiana.	5,900		1-41
(7) Salem, Indiana.	11,700	2.510	1-31
(7) Salem, Indiana.	6,900		
(7) Bloomington, Indiana.	4,200		1-14
(7) Bloomington, Indiana.	5,700	2.460	1-19
(7) Bloomington, Indiana.	8,000		1-33
(7) Romana, Indiana.	7,000	2.480	1-39
(7) Bedford, Indiana.	4,600	2.470	1-23
(7) Bedford, Indiana.	14,000		
(7) Bedford, Indiana.	6,500		1-24
(7) Salem, Indiana.	8,900	2.510	1-31

* See p. 206 for references.

TABLE III (CONTINUED).

Location of Stone.	Comparative Strength in Pounds per Square Inch.	Specific Gravity.	Ratio of Absorption.
MARBLE.			
(1) Rutland, Vermont.	11,892		
(1) Rutland, Vermont.	13,864		
(1) Mountain, Vermont.	12,833		
(1) Sutherland Falls, Vermont.	16,156		
(1) DeKalb, New York.	13,733		
(1) DeKalb, New York.	10,478		
(1) DeKalb, New York.	12,004		
(1) DeKalb, New York.	13,772		
(1) Colton, California.	17,783		
(1) Canaan, Connecticut.	5,812		
(8) St. Joe, Arkansas.	17,835	2.712	0.34
(8) St. Joe, Arkansas.	10,447	2.697	0.33
(8) St. Joe, Arkansas.	11,265	2.707	0.25
(8) Marble City, Arkansas.	8,894	2.691	0.57
(8) Marble City, Arkansas.	10,381	2.689	0.49
(8) Rhodes Mill, Arkansas.	14,400	2.711	0.29
(8) St. Joe, Arkansas.	6,728	2.693	0.37
(8) St. Joe, Arkansas.	6,935	2.675	0.56
(8) Montgomery County, Pennsylvania.	13,700		
(8) Dorset, Vermont.	7,612	2.635	0.58
(8) Cararra, Italy.	12,156	2.690	
(9) Georgia.	10,000		
(9) Georgia.	13,100	2.763	
(9) Georgia.	11,400	2.717	
(9) Georgia.	12,000	2.707	
(9) Georgia.	10,900	2.734	
(9) Georgia.	10,800		

(1) Wisconsin Geological and Natural History Survey, Bulletin o. 4, Building and Ornamental Stones, pp. 309-403, by E. R. Buckley.

(2) Geol. and Nat. Hist. Sur. of Minn., final report, Vol. I, pp. 196-200.

(3) Ann. Rep. Ark. Geol. Survey, Vol. II, 1890, pp. 44 to 50, by J. F. Williams.

(4) Tests of Metals, Government Rep., 1895, pp. 319-320.

(5) Appendix Ann. Rep., Pa. State College, 1896, p. 30 (Brownstones).

(6) Twentieth Ann. Rep., on the Geology and Natural Resources of Indiana, p. 323.

(7) Twenty-first Ann. Rep., on the Geology and Natural Resources of Indiana, pp. 313-315.

(8) Ann. Rep. Ark. Geol. Survey, Vol. IV, 1890, p. 210, by T. C. Hopkins.

(9) Geological Survey of Georgia, Bulletin No. 1, p. 81.

thickness. Some of these machines are so arranged that a channel is cut on each side at the same time. When these channels have been cut, holes are cut horizontally across the bed and the stone split loose by wedges.

Sandstone-quarrying is carried on considerably by channeling-machines, although very many quarries are still operated by blasting out large blocks and cutting them to shape afterwards,

although some damage may result to the stone from the force of the blasts. The method of working quarries is stated quite fully in the Washington Geological Survey as follows:

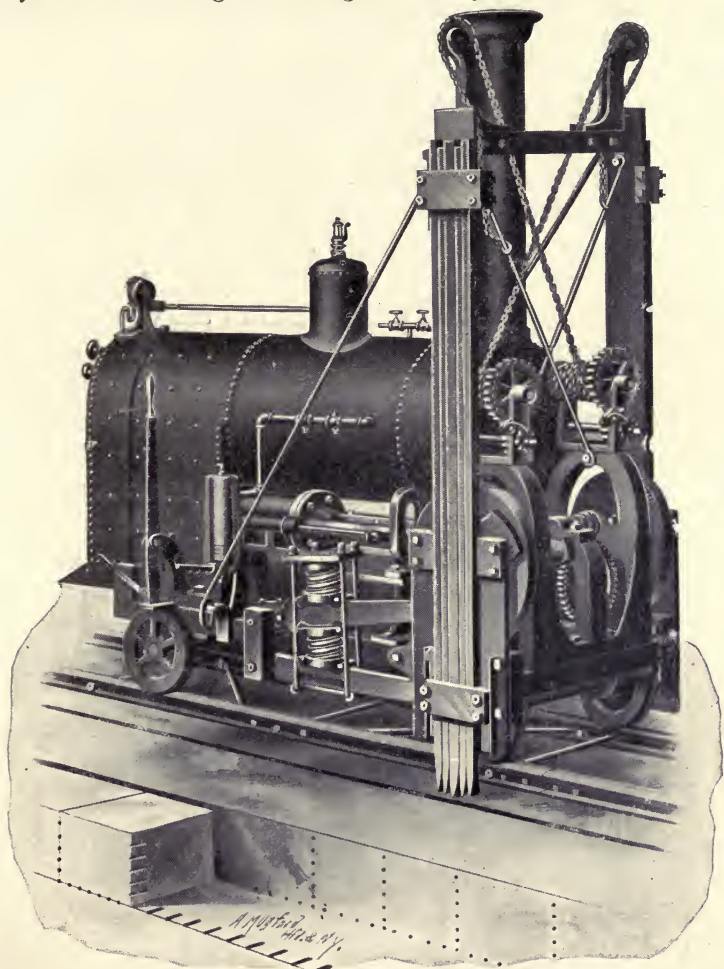


FIG. 124.—WARDWELL CHANNELER.

“The quarrying of marbles, limestones, and some sandstones at the present time is done quite largely by the use of channeling-machines of some kind (Fig. 124), while, in the harder igneous rocks such as granite, explosives are quite largely used for break-

ing the rock loose, after which the large masses are split and worked into sizes by hand. In the opening of a quarry in which channeling-machines are to be used, the usual thing to do is to remove the débris overlying the stone to be quarried and secure a comparatively level floor of the same size that it is desired to make the quarry. When this is done the channeling-machine is put to work and a series of channels the required depth and distance apart are cut, and one of the blocks loosened on the under side in some manner, usually by wedging, and then lifted out or it may be removed by blasting. After the first block is removed the others may be loosened on the under side either by gadding or by means of wedges. Then another layer is begun and removed in exactly the same manner, and in this way the quarry floor may be carried down almost any depth provided the stone continues.

“In some quarries what is known as the step or bench system is used and consists in having a ledge of varying width at the back wall each time instead of taking out an entire layer of the quarry floor. This will give to the back part of the quarry the appearance of a set of steps. If the quarry is to be worked after this plan the bar-channeler is probably the best one to purchase, as it is much more easily moved from bench to bench. In the case of quarries worked by hand, either one of the above plans may be followed.

“While many machines have been invented for cutting and dressing stone, still the same slow hand processes that were in use hundreds of years ago are still quite largely used. Large masses of the stone are loosened by means of powder and then these are split into blocks of the required sizes by what is known as the plug-and-feather method. This method consists in drilling a series of holes about three-fourths of an inch in diameter and a few inches deep along a line where it is desired to split the stone. Into each one of these holes are placed two pieces of soft half-round iron called ‘feathers,’ and between these a steel wedge or ‘plug’ is placed. The quarryman then takes a hammer and moves along this line, striking alternately each one of these wedges until

the stone splits and falls apart along this line. There is considerable knack in the splitting of various kinds of stone and it consists simply in being able to take advantage of the rift and grain of a stone, and it is surprising how readily some persons will work a stone into the desired shape, while others can hardly work it into any shape at all.

"In some cases stone is cut to the proper sizes in the quarry by means of channelers, steam-drills, and portable saws, but in most cases marbles, limestones, and sandstones are cut into the desired shapes after leaving the quarry and going to the mill. Usually the stone is taken out of the quarry in large blocks and then taken to the mill, where it is usually cut into the required dimensions by means of saws, and if it is to be carved or polished this is done here, and, in fact, the stone is finished ready for its place in the building.

"Most of the cutting to sizes is done by sawing (Fig. 125). This sawing is done principally by means of gang-saws which consist of a number of toothless blades of soft iron fastened in a frame in a horizontal position and this frame so arranged that it can be moved backward and forward continuously. The stone to be sawed is brought under these saws and the blades set for the required thickness of the stone and then the machine set in motion. The cutting is done principally by sand or some substitute for it which, along with water, is supplied to the saw-blades. The water softens the stone, aids in carrying the sand to the saw. The saw may be of almost any length and the frame may contain any number of blades. The blades are usually about $\frac{1}{8}$ inch in thickness and about 4 inches wide. In the latest patterns the frames are lowered automatically as the saws cut into the stone.

"The rate of cutting by these saws varies with the stone, being much faster in some kinds than in others, as, for instance, the rate for the Tenino sandstone is from 1 to 2 feet per hour, while in the serpentine, which is a much softer material, at the United States quarry the rate is not more than from 4 to 6 inches per hour.

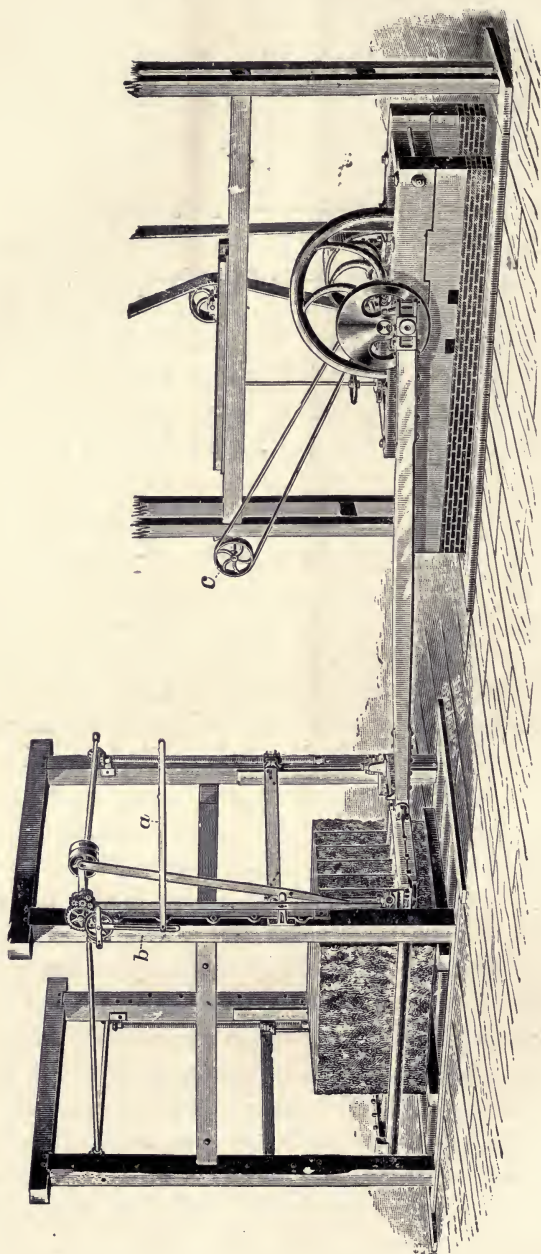


FIG. 125.—STONE SAWING-MACHINE.

"The kind of power used for driving these saws varies and may be steam, electricity, or water-power, and in Washington all three are used. Steam, however, is the one most commonly used, but is much more expensive than water-power.

"Machines of various kinds for planing and dressing marbles have been constructed and they are said to work very satisfactorily, producing a surface equal to a sand-rubbed finish and saving much labor and expense in the finishing of marbles.

"Up to the present time, however, no hing of this kind is being used in this State.

"Lathes of various kinds and sizes are in use in the mills for turning marbles and serpentines. The lathes are used for turning out columns of marble and vases and ornaments of various kinds from marbles and serpentines. These lathes are practically the same as those used for the turning of wood and iron. After the desired shape has been obtained and while the column or whatever it be is still in the lathe the polisher is brought into use and it is finished before it is removed from the lathe.

"For the rubbing of the stone smooth, preparatory to polishing, the most common contrivance is pe.haps what is known as the rubbing-bed. This consi.ts of a heavy cast-iron plate which revolves in a horizontal plane. The plate is revolved by means of a perpendicular shaft through the center, which is geared to the power, this gearing in some cases being above the bed, while in others it may be below. Above the revolving bed are a number of fixed arms extending from the center to the outside of the bed and just high enough above it so as not to touch it. The slabs and blocks of marble are placed on this rubbing-bed, and these arms prevent their revolving with it when it is put in motion. Onto this plate are then put sand or some other abrasive, and water. A large number of pieces may be placed on one bed at the same time and rubbed at much less expense than they could be by hand."

As to the general design of piers, very little further need be said than what has been given in the preceding chapter with reference to the cross-section that produces the least resistance in

a stream, and the quotation from Morison as to the form of pier adopted in the various large bridges with which he was connected.

After the loads to be carried by the pier and the weight of the pier itself have been fully determined, the size of the base of the pier can be easily determined so as to keep the pressure upon the foundation bed inside of proper limits. The unbalanced pressure, due to wind and current, must be taken account of, that from wind being calculated by the ordinary methods of moments, while the force of the current may be calculated from the discussion given in Weisbach's "Mechanics."

Should a particularly large base be required, it will be necessary to offset the courses of the pier by putting in several steps, and these offsets may be calculated by the ordinary formula for transverse loading of beams. Where the pressure on the foundation bed per square foot amounts to two tons, Baker gives the following coefficients by which to multiply the thickness of the masonry course to get the offset: for granite 1.5, for limestone and sandstone 1.3, and concrete in the proportion of 1, 3, and 5, 0.3. As this is the most usual unit pressure, this will be a sufficient guide in ordinary cases. In large piers where caissons and cribs are used, they are made of the proper size to properly distribute the bearing upon the foundation bed.

CHAPTER XIV.

CEMENT AND CONCRETE.

THE use of some cementing material in building construction has been practiced from the earliest times, but the real intelligent employment of cement may be said to date back only as far as the time of the Roman Empire, and Vitruvius in his writings explains fully the methods of its use and the quality of the materials that should be used in making mortar and concrete, although his ideas as to the cause of the cementing action, while describing the results that took place, were very far from the truth, as he had no conception of the chemical processes involved.

The earlier kinds of cement were natural mixtures or deposits of rock which when put through the manufacturing process had the proper composition to form cement. The great trouble, however, with cement of this kind is the lack of uniformity in the composition of the rock itself, so that while one lot of cement might be first-class, another lot made from identically the same quarry might be far from good.

The first real Portland cement that was ever manufactured, so far as known, was under a patent taken out by John Aspdin, of Leeds, England, in 1824, while the first Portland cement manufactured in Germany was made near Stettin in 1852. The manufacture of Portland cement was begun in the United States about 1875, and at the present time there are extensive manufactories in practically every portion of the United States, producing cement in many cases better than the imported. Large amounts of natural cement are manufactured in the United States, and they are first-class for use in making mortar for masonry work,

in making concrete for concrete filling of rock-faced piers, for concrete that is used in foundation pits, and in any location where the concrete is not exposed to abrasion or the weather. But where a first-class piece of work is desired and one that is exposed, nothing should be used in concrete but Portland cement.

Natural deposits of chalk and clay must be found so that they will yield a product containing approximately 62.2 per cent. of lime, 28.2 per cent. of silica, and 9.6 per cent. of alumina, or about one-third of the alumina can be replaced by ferric oxide, giving a cement with a composition of 61.7 per cent. lime, 27.4 per cent. of silica, 7.5 per cent. alumina, and 3.4 per cent. of ferric oxide. As a matter of fact, however, the European cements contain not only the above elements, but small quantities of magnesia, potash, soda, sulphuric acid, sand, carbonic acid, and water.

The process of manufacture of Portland cement does not concern the engineer as a feature of the construction of foundations, so that the details will not be gone into here, but reference may be made to "Portland Cement," by C. D. Jameson, M. Am. Soc. C. E.

When the process of manufacture has been properly carried out, up to the point of grinding, it is necessary that it should be ground with the greatest of care, as upon this depends very largely its value as a cementing substance, and the finer it is ground the more quick-setting it will be—the usual fineness being so that not more than 5 per cent. will remain upon a sieve of 2,500 meshes per inch. In some factories after the cement is ground it is sifted to insure the output being of proper fineness, but as this is an extra expense and apt to cause carelessness among the employés, it is better to be sure that the machinery grinds it properly in the first instance. Nearly all the large manufactories now have storehouses of large capacity in which the cement is stored for some time before shipment, which very greatly improves the quality of the cement. One of the most prominent American brands was shipped out for some years without having been stored previous to packing, as the demand was so great as to make it necessary to ship immediately, but it was found that a considerable amount

of free lime was present and caused the cement to be so quick-setting as to be rejected on important work, so that the manufacturers were compelled to build storehouses and pack it for shipment after it had been stored for some time.

Many engineers require cement to be emptied out from the barrels or bags into a storehouse at the site of the work and become thoroughly mixed and aged before use, but in this country it is usually impracticable to do this, owing to the speed with which work is pushed through. The cement is packed for shipment in barrels, the shipping weight of which is practically 400 pounds gross. These barrels are lined with waterproof paper or keep the cement dry. It is very often packed in paper or burlap sacks containing from one-fourth to one-third of a barrel in each sack. This method of packing is all right if the cement is to be used soon after packing.

For many years the English cements had such a reputation that they were more used than any other; but German manufacturers, realizing that this prestige had caused the English to become somewhat careless, supplanted to a large extent the English cements by simply guaranteeing their product, thus forcing the manufacturers of other countries to turn out a product of greater superiority.

At the present time the methods of testing cements are so uniform that it is only necessary for the makers to furnish a cement that will stand the tests, to have any brand of Portland cement accepted on important work. One of the standard testing-machines is shown in Fig. 126.

Probably the most used specification is that recommended by the Committee on the Testing of Cement of the American Society of Civil Engineers, and a very complete specification of this character is given in the appendix from the specifications for the Topeka Bridge constructed by Edwin Thacher, M. Am. Soc. C. E. But the same engineer has more recently prepared General Specifications for Concrete Steel Bridges, revising the Specifications for Cement, and they are quoted in full as being the most valuable specifications of this character published:

"All foundations shall be shown on plans, and conform to the dimensions marked thereon.

"Foundations on rock shall be prepared by removing all sand, mud, or other soft material, and by excavating the bed-rock in such manner as may be described or shown on drawings.

"Foundations on hard-pan, gravel, gravel and clay, cemented



FIG. 126.—CEMENT TESTING-MACHINE.

sand, or other material intended to carry the load without piles, shall be excavated to the depth shown on plans.

"Foundations on piles, when not otherwise described, shall be inclosed in a permanent coffer-dam or crib, and be excavated to the depths shown on plans, and the piles shall be driven after the excavations are made. The spaces between the piles shall be filled with concrete, and in case it is found necessary to lay the concrete under water proper appliances must be used to insure its being deposited with as little injury as possible.

"The piles shall be oak, yellow pine, or other wood that will stand the blow of the hammer, straight, sound, and cut from

live timber; trimmed close, cut off square at the butt, and have all bark taken off.

"The piles shall not be less than 12 inches nor more than 16 inches in diameter at the large end, nor less than 10 inches in diameter at the small end, for piles having a length of 30 feet and under; for greater lengths the diameter of small end may be reduced 1 inch for each 10 feet of additional length down to a minimum of 7 inches.

"The piles shall not be loaded with a weight greater than given by the following formula:

$$L = 2wh \div (S + 1),$$

in which L = safe load in pounds, w = weight of hammer in pounds, h = fall of hammer in feet, S = last penetration in inches.

"The number and arrangement of the piles for each foundation shall be shown on plans, and they shall be sawed off at the elevations shown.

"The following values shall be used in calculations:

Modulus of elasticity of concrete.....	1,400,000 lbs.
Maximum compression per square inch on concrete.....	500 "
Maximum shear per square inch on concrete.....	100 "
Maximum tension " " " " "	50 "

The above to be exclusive of temperature stresses.

"The cement shall be a true Portland cement, made by calcining a proper mixture of calcarious and clayey earths; and if required the contractor shall furnish a certified statement of the chemical composition of the cement and the raw materials from which it is manufactured.

"The fineness of the cement shall be such that at least 99 per cent. will pass through a sieve of 50 meshes per lineal inch, at least 90 per cent. will pass through a sieve of 100 meshes per lineal inch, and at least 70 per cent. will pass through a sieve of 200 meshes per lineal inch.

"Samples for testing may be taken from each and every barrel delivered unless otherwise specified. Tensile tests will be made on specimens prepared and maintained until tested at a tempera-

ture of not less than 60 degrees Fahrenheit. Each specimen will have an area of one square inch at the breaking section, and after being allowed to harden in moist air for 24 hours will be immersed and maintained under water until tested.

"The sand used in preparing the test specimens shall be clean, sharp, crushed quartz, retained on a sieve of 30 meshes per square inch, and passing through a sieve of 20 meshes per square inch.

"No more than 23 to 27 per cent. of water shall be used in preparing the test specimens of neat cement, and in the case of test specimens of one cement and three sand no more than 11 or 12 per cent. of water by weight shall be used.

"Specimens prepared from neat cement shall after seven days develop a tensile strength of not less than 450 pounds per square inch. Specimens prepared from a mixture of one part cement and three parts sand, parts by weight, shall after seven days develop a tensile strength of not less than 160 pounds per square inch, and not less than 220 pounds per square inch after twenty-eight days. Specimens prepared from a mixture of one part cement and three parts sand, parts by weight, and immersed after twenty-four hours in water maintained at 176 degrees Fahrenheit, shall not swell nor crack, and shall after seven days develop a tensile strength of not less than 160 pounds per square inch.

"Cement mixed neat with about 27 per cent. of water to form a stiff paste, shall after 30 minutes be appreciably indented by the end of a wire one-twelfth inch in diameter loaded to weigh one-quarter pound. Cement made into thin plates on glass plates shall not crack, scale nor warp under the following treatment: Three parts will be made and allowed to harden in moist air at from 60 to 70 degrees Fahrenheit; one of these will be subjected to water-vapor at 176 degrees Fahrenheit for three hours, after which it shall be immersed in hot water for forty-eight hours, another shall be placed in water at from 60 to 70 degrees Fahrenheit, and the third shall be left in moist air.

"All cement shall be kept housed and dry until wanted in the work.

"The concrete shall be composed of clean hard broken stone,

or gravel with irregular surface, clean sharp sand, and cement, mixed in the proportions hereafter specified. Whenever the amount of work to be done is sufficient to justify it, approved mixing-machines shall be used. The ingredients shall be placed in the machine in a dry state, and in the volumes specified and be thoroughly mixed, after which clean water shall be added and the mixing continued until the wet mixture is thorough and the mass uniform. No more water shall be used than the concrete will bear without quaking in ramming. The mixing must be made as rapidly as possible and the batch deposited in the work without delay.

“If the mixing is done by hand, the cement and sand shall first be thoroughly mixed dry in the proportions specified. The stone previously drenched with water shall then be deposited on this mixture. Clean water shall be added and the mass be thoroughly mixed and turned over until each stone is covered with mortar, and the batch shall be deposited without delay, and be thoroughly rammed until all voids are filled. The grades of concrete to be used are as follows: For the arches between skew-backs—one part Portland cement, two parts sand, and four parts broken stone, or gravel, that will pass through a $1\frac{1}{4}$ -inch ring.

“For the foundations, abutments, piers, and spandrels—one part Portland cement, four parts sand, and eight parts broken stone, or gravel, that will pass through a 2-inch ring.

“If concrete facing is used, it shall be composed of one part Portland cement, and two and one-half parts sand, and shall have a thickness of at least 1 inch on all arch soffits, arch faces, abutments, piers, spandrels, or other exposed surfaces.

“There must be no definite plane or surface of demarkation between the facing and the concrete backing. The facing and backing must be deposited in the same layer, and be well rammed in place at the time same. If the arch faces, quoins, or other exposed surfaces are marked to represent masonry, such division marks shall be made by triangular strips 2 inches wide and 1 inch deep fastened to the casing in perfectly straight and parallel lines, and all projecting corners will be beveled to correspond.

"No plastering will be allowed on the exposed faces of the work, but the inside faces of the spandrel walls covered by the fill may be plastered with mortar having the same composition as specified for facing.

"All keystones, brackets, consoles, dentils, pedestals, hand-railing posts and panels, and other ornamental work when used, also curbs and gutters, shall be of the designs shown on plans, and be molded in suitable molds. The mortar for at least one inch thick shall consist of one part Portland cement and two and one-half parts sand, and when the size of the molding will admit, the interior may be composed of concrete of the same composition as specified for the arches. When pedestals, posts, or panels carry, lamp-posts, a 4-inch wrought-iron pipe shall be built into the concrete from top to bottom, and at bottom shall be connected with a 3-inch pipe extending under the sidewalk and connected with gas-pipe or electric-wire conduit. The pipes shall have no sharp bends, all changes in direction being made by gentle curves.

"During warm and dry weather, all newly built concrete shall be well sprinkled with water for several days, or until it is well set.

"The volumes of cement, sand, and broken stone in all mixtures of mortar or concrete used in the work shall be measured loose.

"In connecting concrete already set with new concrete the surface shall be cleaned and roughened, and mopped with a mortar composed of one part Portland cement and one part sand, to cement the parts together.

"The concrete for the arches shall be started simultaneously from both ends of the arch, and be built in longitudinal sections wide enough to inclose at least two steel ribs, and of sufficient width to constitute a day's work. The concrete shall be deposited in layers, each layer being well rammed in place before the previously deposited layer has had time to partially set. The work shall proceed continuously day and night if necessary to complete each longitudinal section. These sections while being built shall be held in place by substantial timber forms, normal to the center-

ing and parallel to each other, and these forms shall be removed when the section has set sufficiently to admit of it. The sections shall be connected as specified and also by steel clamps or rib connections built into the concrete."

The average requirements of the United States Government Engineers are that 95½ per cent. shall pass through a 2,500-mesh sieve and 84 per cent. through a 10,000-mesh sieve. This requirement is considerably lower than is called for by many of the cities in the United States, the average of nine cities taken for this purpose requiring 97 per cent. to pass through a 2,500-mesh sieve and 89 per cent. through a 10,000-mesh sieve. The United States Government Engineers require the cement to test up to 402 pounds per square inch for seven-day tests of neat cement, and up to 119 pounds for a seven-day test of 3 to 1 briquettes; while the same nine cities mentioned above show an average requirement of 388 pounds per square inch for seven-day tests of neat cement, and 134 pounds for seven-day tests of 3 to 1 briquettes.

The proportion of concrete and the methods of depositing it have been fully covered in the previous chapter, but the practical use of cement makes it desirable to have tables of concrete mixtures of various proportions, which give the amount of cement in barrels, the amount of sand in yards, and the amount of broken stone or gravel in yards, to use to make one yard of tamped concrete.

One of the first tables of this sort to be published, which had been checked by practice, was in Vol. 42 of the Transactions of the American Society of Civil Engineers, in a discussion by the author, of a paper by Geo. W. Rafter, M. Am. Soc. C. E., on The Theory of Concrete. More complete tables of this kind have been worked out from theory, experiment, and practice by Edwin Thacher, M. Am. Soc. C. E., and with his permission are reprinted here.

The discussion by the author on the amount of material in different concretes is given in full, as it forms an interesting comparison with the tables prepared by Thacher. The column

TABLE IV.—MATERIAL FOR CONCRETE.

Concrete with "Hazelnut" Stone.						Concrete with Stone $2\frac{1}{4}$ " and Under.					
Proportions of Mixture.			Required for 1 Cubic Yard.			Proportions of Mixture.			Required for 1 Cubic Yard.		
Cem-ent.	Sand.	Stone.	Cem-ent, Barrels	Sand, Cubic Yards.	Stone, Cubic Yards.	Cem-ent.	Sand.	Stone.	Cem-ent, Barrels	Sand, Cubic Yards.	Stone, Cubic Yards.
I	I	2.0	2.57	0.39	0.78	I	I	2.0	2.63	0.40	0.80
I	I	2.5	2.29	0.35	0.70	I	I	2.5	2.34	0.36	0.89
I	I	3.0	2.06	0.31	0.94	I	I	3.0	2.10	0.32	0.96
I	I	3.5	1.84	0.28	0.98	I	I	3.5	1.88	0.29	1.00
I	1.5	2.5	2.05	0.47	0.78	I	1.5	2.5	2.09	0.48	0.80
I	1.5	3.0	1.85	0.42	0.84	I	1.5	3.0	1.90	0.43	0.87
I	1.5	3.5	1.72	0.39	0.91	I	1.5	3.5	1.74	0.40	0.93
I	1.5	4.0	1.57	0.36	0.96	I	1.5	4.0	1.61	0.37	0.98
I	1.5	4.5	1.43	0.33	0.98	I	1.5	4.5	1.46	0.33	1.00
I	2.0	3.0	1.70	0.52	0.77	I	2.0	3.0	1.73	0.53	0.79
I	2.0	3.5	1.57	0.48	0.83	I	2.0	3.5	1.61	0.49	0.85
I	2.0	4.0	1.46	0.44	0.89	I	2.0	4.0	1.48	0.45	0.90
I	2.0	4.5	1.36	0.42	0.93	I	2.0	4.5	1.38	0.42	0.95
I	2.0	5.0	1.27	0.39	0.97	I	2.0	5.0	1.29	0.39	0.98
I	2.5	3.5	1.45	0.55	0.77	I	2.5	3.5	1.48	0.56	0.79
I	2.5	4.0	1.35	0.52	0.82	I	2.5	4.0	1.38	0.53	0.84
I	2.5	4.5	1.27	0.48	0.87	I	2.5	4.5	1.29	0.49	0.88
I	2.5	5.0	1.19	0.46	0.91	I	2.5	5.0	1.21	0.46	0.92
I	2.5	5.5	1.13	0.43	0.94	I	2.5	5.5	1.15	0.44	0.96
I	2.5	6.0	1.07	0.41	0.97	I	2.5	6.0	1.07	0.41	0.98
I	3.0	4.0	1.26	0.58	0.77	I	3.0	4.0	1.28	0.58	0.78
I	3.0	4.5	1.18	0.54	0.81	I	3.0	4.5	1.20	0.55	0.82
I	3.0	5.0	1.11	0.51	0.85	I	3.0	5.0	1.14	0.52	0.87
I	3.0	5.5	1.06	0.48	0.89	I	3.0	5.5	1.07	0.49	0.90
I	3.0	6.0	1.01	0.46	0.92	I	3.0	6.0	1.02	0.47	0.93
I	3.0	6.5	0.96	0.44	0.95	I	3.0	6.5	0.98	0.44	0.96
I	3.0	7.0	0.91	0.42	0.97	I	3.0	7.0	0.92	0.42	0.98
I	3.5	5.0	1.05	0.56	0.80	I	3.5	5.0	1.07	0.57	0.82
I	3.5	5.5	1.00	0.53	0.84	I	3.5	5.5	1.02	0.54	0.85
I	3.5	6.0	0.95	0.50	0.87	I	3.5	6.0	0.97	0.51	0.89
I	3.5	6.5	0.92	0.49	0.91	I	3.5	6.5	0.93	0.49	0.92
I	3.5	7.0	0.87	0.47	0.93	I	3.5	7.0	0.89	0.47	0.95
I	3.5	7.5	0.84	0.45	0.96	I	3.5	7.5	0.85	0.45	0.98
I	3.5	8.0	0.80	0.42	0.97						
I	4.0	6.0	0.90	0.55	0.82	I	4.0	6.0	0.92	0.56	0.84
I	4.0	6.5	0.87	0.53	0.85	I	4.0	6.5	0.88	0.53	0.87
I	4.0	7.0	0.83	0.51	0.89	I	4.0	7.0	0.84	0.51	0.90
I	4.0	7.5	0.80	0.49	0.91	I	4.0	7.5	0.81	0.50	0.93
I	4.0	8.0	0.77	0.47	0.93	I	4.0	8.0	0.78	0.48	0.95
I	4.0	8.5	0.74	0.45	0.95	I	4.0	8.5	0.76	0.46	0.98
I	4.0	9.0	0.71	0.43	0.97						
I	5.0	9.0	0.66	0.50	0.90	I	5.0	9.0	0.67	0.52	0.93
I	5.0	10.0	0.62	0.47	0.95	I	5.0	10.0	0.63	0.48	0.96
I	6.0	11.0	0.55	0.51	0.93	I	6.0	11.0	0.56	0.52	0.94
I	6.0	12.0	0.52	0.48	0.95	I	6.0	12.0	0.54	0.49	0.98
I	7.0	13.0	0.47	0.50	0.93	I	7.0	13.0	0.48	0.51	0.95
I	7.0	14.0	0.45	0.48	0.96	I	7.0	14.0	0.46	0.49	0.98

TABLE V.—MATERIAL FOR CONCRETE (CONTINUED).

Concrete with 2½ Inch Stone.						Concrete with Gravel ¼" and Under.					
Proportions of Mixture.			Required for 1 Cubic Yard.			Proportions of Mixture.			Required for 1 Cubic Yard.		
Cem-ent.	Sand.	Stone.	Cem-ent, Barrels	Sand, Cubic Yards.	Stone, Cubic Yards.	Cem-ent.	Sand.	Grav-el.	Cem-ent, Barrels	Sand, Cubic Yards.	Gravel, Cubic Yards.
I	I	2.0	2.72	0.41	0.83	I	I	2.5	2.10	0.32	0.80
I	I	2.5	2.41	0.37	0.92	I	I	3.0	1.89	0.29	0.86
I	I	3.0	2.16	0.33	0.98	I	I	3.5	1.71	0.26	0.91
.....						I	I	4.0	1.55	0.24	0.94
I	1.5	2.5	2.16	0.49	0.82	I	1.5	3.0	1.71	0.39	0.78
I	1.5	3.0	1.96	0.45	0.89	I	1.5	3.5	1.57	0.36	0.83
I	1.5	3.5	1.79	0.41	0.96	I	1.5	4.0	1.46	0.33	0.88
I	1.5	4.0	1.64	0.38	1.00	I	1.5	4.5	1.34	0.31	0.91
.....						I	1.5	5.0	1.24	0.28	0.94
I	2.0	3.0	1.78	0.54	0.81	I	2.0	3.5	1.44	0.44	0.77
I	2.0	3.5	1.66	0.50	0.88	I	2.0	4.0	1.34	0.41	0.81
I	2.0	4.0	1.53	0.47	0.93	I	2.0	4.5	1.26	0.38	0.86
I	2.0	4.5	1.43	0.43	0.98	I	2.0	5.0	1.17	0.36	0.89
.....						I	2.0	6.0	1.03	0.31	0.94
I	2.5	3.5	1.51	0.58	0.81	I	2.5	4.0	1.24	0.47	0.75
I	2.5	4.0	1.42	0.54	0.87	I	2.5	4.5	1.16	0.44	0.80
I	2.5	4.5	1.33	0.51	0.91	I	2.5	5.0	1.10	0.42	0.83
I	2.5	5.0	1.26	0.48	0.96	I	2.5	5.5	1.03	0.39	0.86
I	2.5	5.5	1.18	0.44	0.99	I	2.5	6.0	0.98	0.37	0.89
.....						I	2.5	7.0	0.88	0.33	0.93
I	3.0	4.0	1.32	0.60	0.80	I	3.0	5.0	1.03	0.47	0.78
I	3.0	4.5	1.24	0.57	0.85	I	3.0	5.5	0.97	0.44	0.81
I	3.0	5.0	1.17	0.54	0.89	I	3.0	6.0	0.92	0.42	0.84
I	3.0	5.5	1.11	0.51	0.93	I	3.0	6.5	0.88	0.40	0.87
I	3.0	6.0	1.06	0.48	0.97	I	3.0	7.0	0.84	0.38	0.89
.....						I	3.0	7.5	0.80	0.37	0.91
.....						I	3.0	8.0	0.76	0.35	0.93
I	3.5	5.0	1.11	0.59	0.85	I	3.5	6.0	0.88	0.46	0.80
I	3.5	5.5	1.06	0.56	0.89	I	3.5	6.5	0.83	0.44	0.82
I	3.5	6.0	1.00	0.53	0.92	I	3.5	7.0	0.80	0.43	0.85
I	3.5	6.5	0.96	0.51	0.95	I	3.5	7.5	0.76	0.41	0.87
I	3.5	7.0	0.91	0.49	0.98	I	3.5	8.0	0.73	0.39	0.89
.....						I	3.5	8.5	0.71	0.38	0.91
.....						I	3.5	9.0	0.68	0.36	0.92
I	4.0	6.0	0.95	0.58	0.87	I	4.0	7.0	0.77	0.47	0.81
I	4.0	6.5	0.91	0.55	0.90	I	4.0	7.5	0.73	0.44	0.83
I	4.0	7.0	0.87	0.53	0.93	I	4.0	8.0	0.71	0.43	0.86
I	4.0	7.5	0.84	0.51	0.96	I	4.0	8.5	0.68	0.42	0.88
I	4.0	8.0	0.81	0.49	0.98	I	4.0	9.0	0.65	0.40	0.89
.....						I	4.0	9.5	0.63	0.38	0.91
.....						I	4.0	10.0	0.61	0.37	0.93
I	5.5	8.0	0.74	0.57	0.91	I	5.0	10.0	0.57	0.43	0.87
I	5.0	9.0	0.70	0.53	0.96	I	5.0	12.0	0.51	0.38	0.92
I	6.0	9.0	0.65	0.59	0.89	I	6.0	12.0	0.48	0.44	0.88
I	6.0	10.0	0.62	0.56	0.93	I	6.0	14.0	0.43	0.40	0.92
I	7.0	11.0	0.54	0.51	0.91	I	7.0	14.0	0.42	0.44	0.88
I	7.0	12.0	0.52	0.55	0.95	I	7.0	16.0	0.38	0.40	0.92

with stone having 0.4 voids agrees very closely with Thacher's table for concrete with $2\frac{1}{2}$ " stone, the agreement being so close as to make the tables practically identical.

"This subject has impressed the speaker as being a very important one, for the reason that in figuring on work he has, as a rule, found that engineers and contractors know only approximately the quantities of cement, sand and broken stone to order for a given number of yards of concrete of certain specified proportions. The most convenient way of determining these quantities is to prepare tables from which the amounts required for a yard of concrete of any proportion may be taken, using only such proportions as have the voids in the stone entirely or more than filled.

"Many writers on this subject have assumed that every concrete, no matter what the proportion may be, has simply a yard of broken stone, without regard to the fact that there is more or less mortar in different proportions. In making up a table of this sort the speaker used the proportions of mortar given in Baker's "Masonry Construction," but found that this did not agree with actual practice. These tables in "Masonry Construction" were made up from theoretical considerations, and it is understood that they are to be revised in a new edition. [This has since been done.]

"The table was made up some years ago by comparison with actual cases, and has been found to give very satisfactory results in practice. The column for broken stone with 0.4 voids will represent closely ordinary broken limestone which chips up more than a harder stone and consequently has less voids; while the column with 0.5 voids represents trap rock, the break in which is more angular. It will be readily seen from the table for Portland cement concrete that certain proportions will not have the voids filled and should not be used.

"For example, with 1 to 3 mortar and with stone having 0.4 voids all the concrete given will have the voids filled, or more than filled, while with 0.5 voids only the 1-3-4 (No. 7) will have the voids filled. The table is made up on the basis of shrinkage

under ramming of about 7 cubic feet, and 3.8 cubic feet of cement to a barrel.

"In a recent paper presented to the society, one of the members stated that concrete was not a fit material for piers of railway bridges, unless a pedestal block of stone was used on which to rest the masonry plates. The speaker would refer that member to the engineers of half a dozen large western roads who use concrete for piers, and allow the bed plates to rest directly upon the concrete. Concrete is certainly better than most stone, except granite, and probably 50 per cent. better than much of the stone commonly used."

TABLE VI.—PORTLAND CEMENT CONCRETE.

Number.	Proportions.	Barrels Cement.	Cubic Yards Sand.	Stone, 0.4 Voids.	Stone, 0.5 Voids.
1.....	1-2-3	1.77	0.51	0.87	1.05
2.....	1-2-3½	1.68	0.49	0.91	1.10
3.....	1-2-4	1.59	0.47	0.95	1.15
4.....	1-2-4½	1.48	0.44	1.00	1.20
5.....	1-2-5	1.39	0.42	1.04	1.26
6.....	1-2-5½	1.30	0.40	1.08	1.30
7.....	1-3-4	1.30	0.57	0.83	1.00
8.....	1-3-4½	1.22	0.54	0.89	1.06
9.....	1-3-5	1.16	0.52	0.92	1.11
10.....	1-3-5½	1.09	0.50	0.97	1.16
11.....	1-3-6	1.04	0.48	1.00	1.20
12.....	1-4-6	1.00	0.55	0.91	1.09
13.....	1-4-6½	0.96	0.53	0.94	1.13
14.....	1-4-7	0.92	0.51	0.97	1.17
15.....	1-4-7½	0.88	0.49	1.00	1.21
16.....	1-4-8	0.83	0.47	1.03	1.25

About 34 cubic feet loose=1 cubic yard rammed.

Labor=½ to 2 cubic yards per man per day.

For machine mixing the new Ransome mixer, or the Smith mixer (Fig. 127) are considered by the author to be among the best for turning out thoroughly and uniformly mixed concrete.

Examples of concrete construction carried out by the author are shown in Figs. 128, 129, 130 and 131.

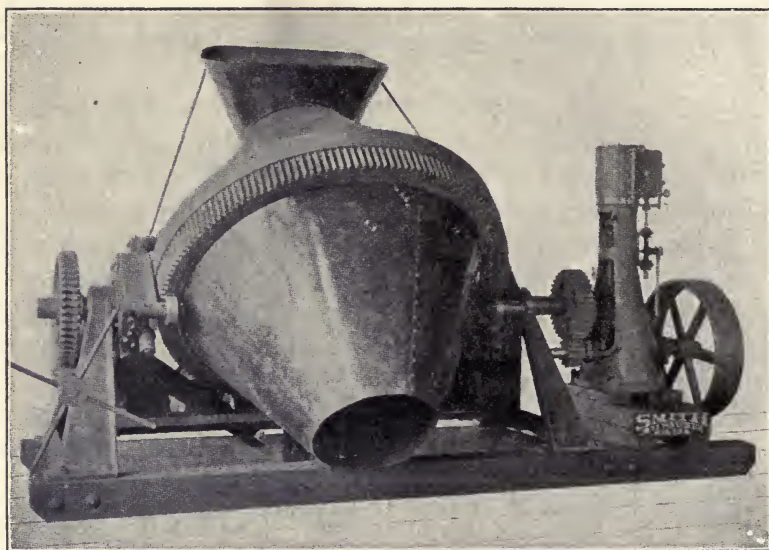


FIG. 127.—SMITH CONCRETE-MIXER.

Copy of Report of the Operations of the Engineering Department of the District of Columbia, under the direction of Major Charles F. Powell, Corps of Engineers, U. S. A., Engineer Commissioner of District of Columbia, year ending June 30, 1896. (Page 194.)

TABLE VII.—TENSILE STRENGTH: 3 PARTS QUARTZ, 1 PART CEMENT.

	7 Days	1 Month	2 Months	3 Months	4 Months	5 Months	6 Months	12 Months
Atlas.....	321	441	441	510	519	525	538	546
Alsen.....	188	310	290	328	385	380	390	366
Dyckerhoff.....	164	175	192	236	257	293	298	323
Hanover.....	205	244	251	277	301	315	315	354
Alpha.....	105	182	310	309	310	295	327	350
Hemmoor.....	159	203	286	301	323	329	314	347
Giant.....	230	275	275	267	296	329	325	327
Porta.....	181	257	305	319	315	322	343	329
Egypt.....	159	205	255	240	285	301	341	394
Henry.....	159	188	229	277	300	320	319	332
Mannheimer.....	193	226	306	329	335	323	343	336
Saylor's.....	135	156	205	203	254	277	289	279



FIG. 128.—PUGET SOUND NAVY YARD.



FIG. 129.—KNOXVILLE CONCRETE ABUTMENT.

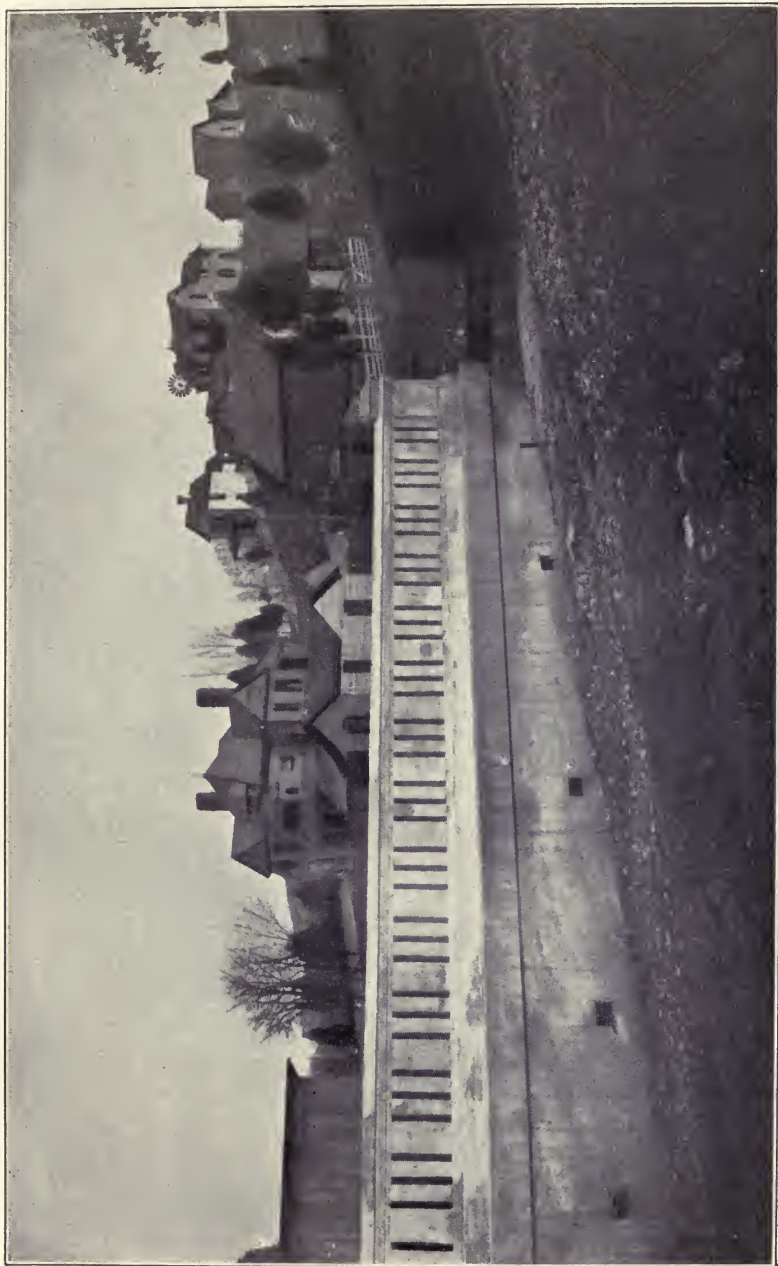


FIG. 130.—KNOXVILLE PARAPETS.

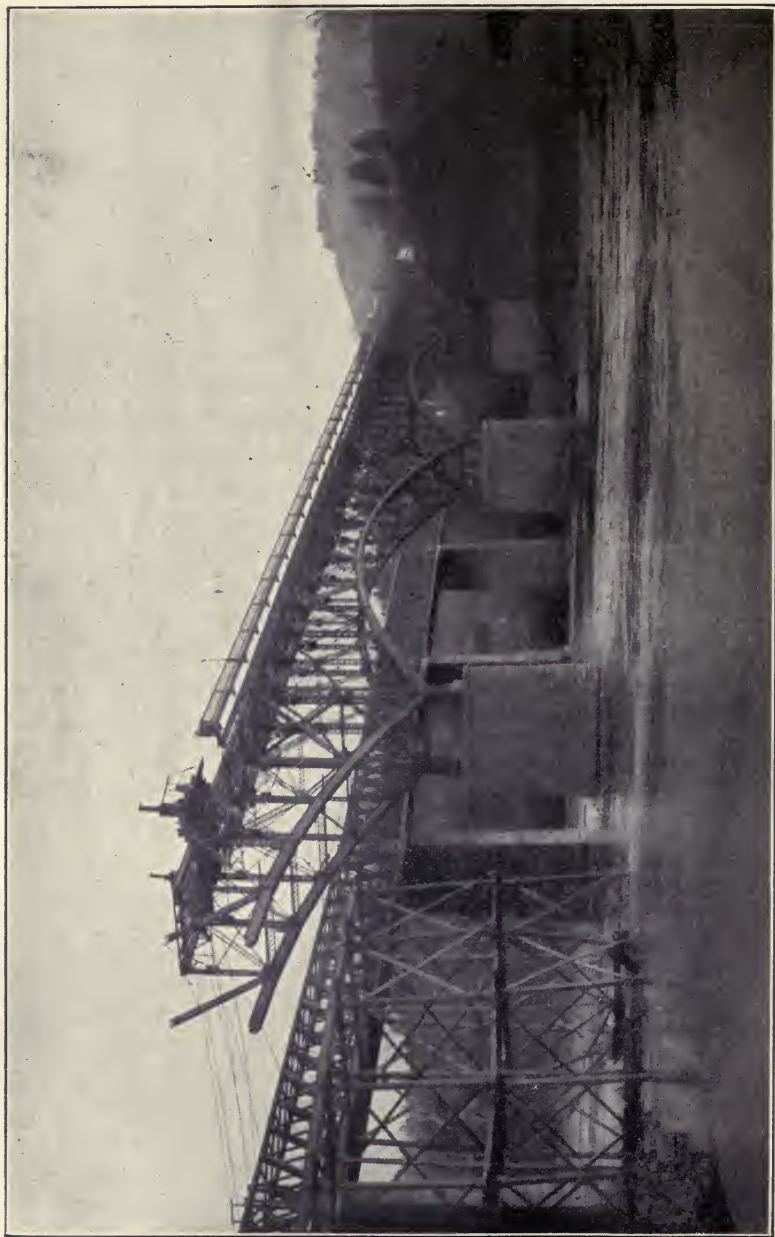


FIG. 131.—KNOXVILLE BRIDGE ERECTION.

Summary of cement tests for tensile strength and fineness made by the Seattle City Engineer's Office to this date, March 10, 1904.

TABLE VIII.—MIXTURE: 1 VOLUME OF CEMENT TO 1 VOLUME OF SAND.

Brand.	No. Briquettes.	Tensile Strength.						Fineness. On No. 100 Sieve, 10,000 Meshes to Sq. Inch.
		8 Days.	31 Days.	3 Months	6 Months	1 Year.	2 Years.	
Alsen.	6	415	480	547	560	634	664	87%
Condor.	6	319	458	555	606	641	703	87.5
Dykerhoff.	7	443	482	561	605	691	...	90
Flying Cask.	12	472	580	604	...	...	...	90
Fortification.	6	309	361	477	479	...	...	93
Germania.	6	308	412	484	510	606	...	92
Golden Gate.	17	297	424	517	537	...	...	84.8
Harcourt.	6	207	300	341	366	424	466	87
Heidelberg.	6	153	240	284	363	393	424	88.5
Hemmoor.	6	306	391	446	504	557	...	87
Hercules.	6	175	289	354	384	448	...	85.3
Josson.	6	259	369	425	475	530	584	88
K. B. & S.	6	273	286	424	...	...	...	87.5
Mannheimer.	6	341	401	470	519	507	585	88
Red Castle.	6	215	261	334	402	488	526	87
Rudersdorf.	6	331	425	481	545	591	...	91
Standard.	18	357	428	573	587	...	...	94.5
Teutonia.	5	330	400	433	475	508	...	85
Trowel.	6	178	252	348	404	475	...	85
Utah.	6	320	461	475	556	592	603	85

CHAPTER XV.

TIMBER PIERS AND TIMBER PRESERVATION.

THE construction of piers of piling and sawed timber is quite common throughout the Western states, and, owing to the cheapness with which they can be built, it is often possible to construct a bridge where, if permanent foundations had to be put in, the expense would be so great as to be prohibitive. Where the piles are driven in the ground outside the water-line they will last from five to seven years, and where they are driven in fresh water they will last for a considerably longer period except that the tops of the piles will very often rot out, have to be cut off, and have blocking substituted. Where such piers are to be constructed in salt water the piling should be protected from the teredo by some process or other, preferably by creosoting, as that will insure against both rotting and the teredo. It is also advisable in piers constructed on land, or in fresh water, to protect the piles in some way, either by creosoting or by coating them with hot carbolineum avenarius. The abutment pier, shown in Figs. 132 and 133, was constructed on the line of the Puget Sound Electric Railway between Seattle and Tacoma, as the end pier to a 200-foot draw span, the bridge and foundations being constructed from plans prepared by the author. The bridge seats come directly over two piles, and these are closely flanked by two additional piles on each side, spaced 2 feet 6 inches from the center line of the trusses each way, while four additional piles are driven in the center of the pier. The piles are capped crosswise with short 12"×12"-caps, and on these are laid the longitudinal 12"×12"-caps which carry the bridge

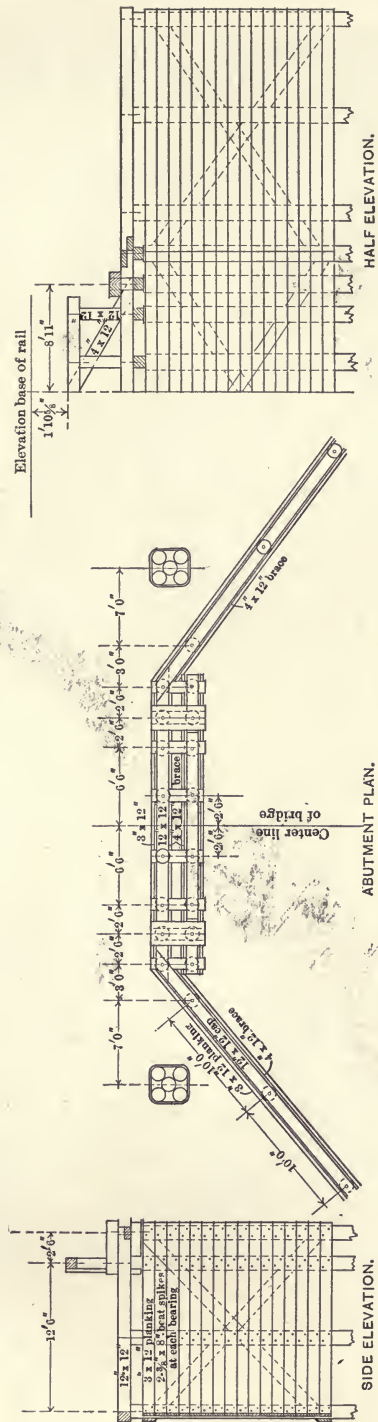


FIG. 132.—PILE PIER, PUGET SOUND ELECTRIC RAILWAY.

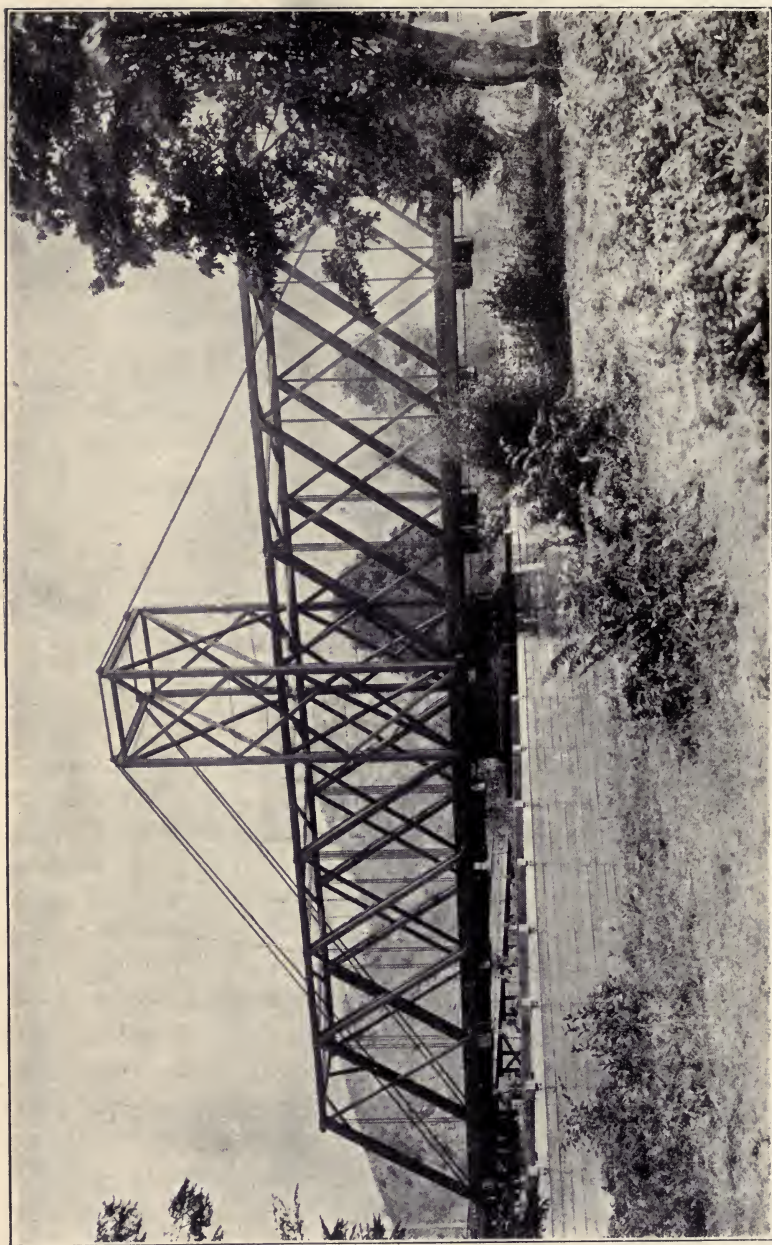


FIG. 133.—DUWAMISH DRAW, PUGET SOUND ELECTRIC RAILWAY.

seats. The piles are braced with 4"×12"-diagonal braces boat-spiked to the piles at each intersection. The same method of construction is used for the wings, and the whole face of the pier and wings is planked up with 3"×12"-planking spiked on, with two 8-inch boat-spikes in each plank at each pile. The piling was of Washington fir with the bark peeled off and treated with hot carbolineum avenarius. These piles were driven to a firm bearing.

The timber was No. 1 merchantable Washington fir, free from sap wind-shake, pitch seams or other imperfections that might impair its strength and durability. In addition to the planking on the face of the pier, to protect it, five-pile dolphins were driven, as shown, 10 feet from the center of the end piles of the pier proper, and wrapped together with wire cables. Carefully constructed as these piers were, it is believed that their life will be from ten to twelve years at the least calculation. The piers, as well as the bridges, were calculated to carry train-loads consisting of Cooper's E-27 Loading. The cost of the piling was seven cents per lineal foot delivered at the bridge site, to which should be added the cost of coating, while the cost of the timber was twelve dollars delivered at the bridge site. The cost of driving the piles was approximately four dollars each for driving and cutting off, and the cost of placing the timber was approximately four dollars per thousand feet board measure. Another pier on the same road is shown in Fig 136.

In the construction of the Raging River bridge and trestle, on the line of the Northern Pacific Railroad, between Seattle and Snoqualmie Falls, it was necessary to cross the stream with a Howe truss span supported on timber towers or piers, a detail of which is shown in Fig. 134. The original trestle and bridge was left standing about thirteen years, or about three years longer than the life of the timber, and fell down during the passage of a log train. The fall was caused primarily by some logs falling off of the train and knocking out some of the batter posts of the trestle, after which the entire trestle collapsed, although it had outlived its usefulness owing to the age of the timber. This

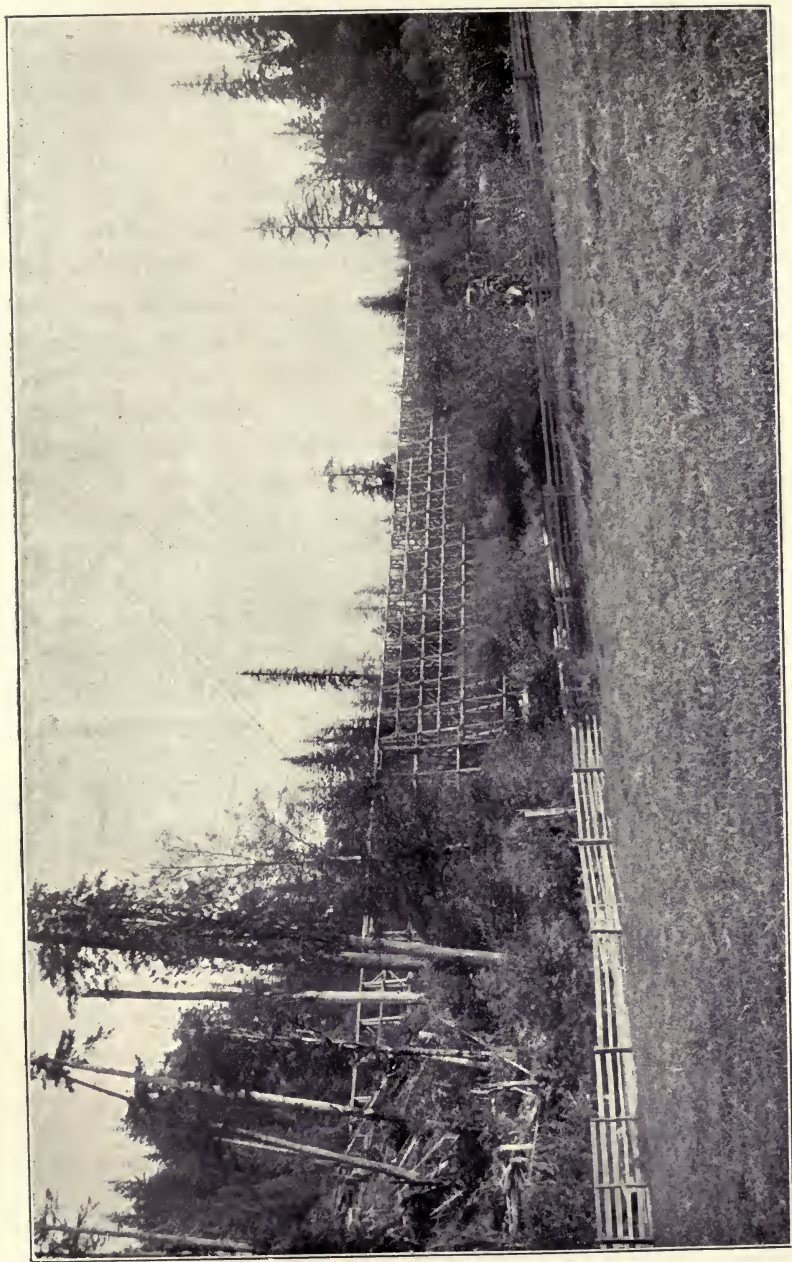


FIG. 135.—RAGING RIVER BRIDGE, NORTHERN PACIFIC RAILWAY.

them, by laying them out on the ground and framing the work complete down to the boring of the bolt holes. The plumb posts were braced together transversely with Howe truss bracing, consisting of timber diagonals formed of two 6"×8"-sticks in one direction and one stick 8"×10" in the other direction. The batter posts were braced to the plumb posts with 5"×12" and 6"×12" girts, and 5"×12" diagonal braces. The plumb posts



FIG. 136.—PIER OF GEORGETOWN BRIDGE.

and batter posts were each formed of two 9"×14"-sticks packed together. Additional batter posts 9"×12" ran to longitudinal girts at half the height of the pier. The truss-rods of the Howe truss bracing varied from 1 inch to 1½ inches in size, while most of the bolts used for fastening the work together were 1 inch. The tower rested upon 10"×14" mud-sills and 9"×16" main-sills with caps of the same size. The ends of all timbers were white-leaded, but none of it was painted or treated in any way. Such a pier as this should be good for from eight to ten years' service for ordinary railroad traffic, although new work of this

character, on even the Western railroads, at the present time is easily constructed of steel.

The designing of timber trestles, which would cover piers of the character under discussion, is most fully treated in Bulletin No. 12 of the U. S. Department of Agriculture, Timber Physics Series, the title of the Bulletin being "Economical Designing of Timber Trestle Bridges." The treatise is the work of A. L. Johnson, C. E.

The strength of bridge and trestle timbers has been fully treated in a report by a committee of the American International Association of Railway Superintendents of Bridges and Buildings on the "Strength of Bridge and Trestle Timbers."

"The test data at hand and the summary of criticisms of leading authorities seem to indicate the general correctness of the following conclusions:

"(1) Of all structural materials used for bridges and trestles, timber is the most variable as to the properties and strength of the different pieces classed as belonging to the same species; hence it is impossible to establish close and reliable limits for each species.

"(2) The various names applied to one and the same species in different parts of the country lead to great confusion in classifying or applying results of tests.

"(3) Variations in strength are generally directly proportional to the density or weight of timber.

"(4) As a rule, a reduction of moisture is accompanied by an increase in strength; in other words, seasoned lumber is stronger than green lumber.

"(5) Structures should be, in general, designed for the strength of green or moderately seasoned lumber of average quality and not for a high grade of well-seasoned material.

"(6) Age and use do not destroy the strength of timber unless decay or season checking takes place.

"(7) Timber, unlike materials of a more homogeneous nature, as iron and steel, has no well-defined limit of elasticity. As a rule, it can be strained very near to the breaking point without

serious injury, which accounts for the continuous use of many timber structures with the material strained far beyond the usually accepted safe limits. On the other hand, sudden and frequently inexplicable failures of individual sticks at very low limits are liable to occur.

“(8) Knots, even when sound and tight, are one of the most objectionable features of timber, both for beams and struts. The full-size tests of every experimenter have demonstrated not only that beams break at knots, but that invariably timber struts will fail at a knot or owing to the proximity of a knot, by reducing the effective area of the stick and causing curly and cross-grained fibers, thus exploding the old practical view that sound and tight knots are not detrimental to timber in compression.

“(9) Excepting in top logs of a tree or very small and young timber, the heart-wood is, as a rule, not as strong as the material farther away from the heart. This becomes more generally apparent, in practice, in large sticks with considerable heart-wood cut from old trees in which the heart has begun to decay or been wind-shaken. Beams cut from such material frequently season check a'ong middle of beam and fail by longitudinal shearing.

“(10) Top logs are not as strong as butt logs, provided the latter have sound timber.

“(11) The results of compression tests are more uniform and vary less for one species of timber than any other kind of test; hence, if only one kind of test can be made, it would seem that a compressive test will furnish the most reliable comparative results.

“(12) Long timber columns generally fail by lateral deflection or ‘buckling’ when the length exceeds the least cross-sectional dimension of the stick by 20; in other words, when the column is longer than 20 diameters. In practice the unit stress for all columns over 15 diameters should be reduced in accordance with the various rules and formulæ established for long columns.

“(13) Uneven end bearings and eccentric loading of columns produce more serious disturbances than are usually assumed.

“(14) The tests of full-size long compound columns, composed of several sticks bolted and fastened together at intervals, show essentially the same ultimate unit resistance for the compound column as each component stick would have if considered as a column by itself.

“(15) More attention should be given in practice to the proper proportioning of bearing areas; in other words, the compressive bearing resistance of timber with and across grain, especially the latter, owing to the tendency of an excessive crushing stress across grain to indent the timber, thereby destroying the fiber and increasing the liability to speedy decay, especially when exposed to the weather and the continual working produced by moving loads.

“The aim of your committee has been to examine the conflicting test data at hand, attributing the proper degree of importance to the various results and recommendations, and then to establish a set of units that can be accepted as fair average values, as far as known to-day, for the ordinary quality of each species of timber and corresponding to the usual conditions and sizes of timbers encountered in practice. The difficulties of executing such a task successfully cannot be overrated, owing to the meagerness and frequently the indefiniteness of the available test data, and especially the great range of physical properties in different sticks of the same general species, not only due to the locality where it is grown, but also to the condition of the timber as regards the percentage of moisture, degree of seasoning, physical characteristics, grain, texture, proportion of hard and soft fibers, presence of knots, etc., all of which affect the question of strength.

“Your committee recommends, upon the basis of the test data at hand at the present time, the average units for the ultimate breaking stresses of the principal timbers used in bridge and trestle constructions shown in the accompanying table.

“In addition to the units given in the table, attention should be called to the latest formulæ for long timber columns, mentioned more particularly in the appendix to this report, which formulæ are based upon the results of the more recent full-size timber-

column tests, and hence should be considered more valuable than the older formulæ derived from a limited number of small-size tests. These new formulæ are Professor Burr's, Appendix I; Professor Ely's, Appendix J; Professor Stanwood's, Appendix K; and A. L. Johnson's, Appendix V; while C. Shaler Smith's formulæ will be better understood after examining the explanatory notes contained in Appendix L. (The formula recommended for use by the author is nearly that of Professor Burr, or $p = 6000 - 70 \frac{l}{d}$, in which p = ultimate strength per square inch, l = length of column in inches, and d = the least dimension of the column in inches. This is for Washington fir or similar timber, between the limits of $20 \frac{l}{d}$ and $60 \frac{l}{d}$.)

"Attention should be called to the necessity of examining the resistance of a beam to longitudinal shearing along the neutral axis, as beams under transverse loading frequently fail by longitudinal shearing in the place of transverse rupture.

"In addition to the ultimate breaking unit stress the designer of a timber structure has to establish the safe allowable unit stress for the species of timber to be used. This will vary for each particular class of structures and individual conditions. The selection of the proper 'factor of safety' is largely a question of personal judgment and experience, and offers the best opportunity for the display of analytical and practical ability on the part of the designer. It is difficult to give specific rules. The following are some of the controlling questions to be considered:

"The class of structure, whether temporary or permanent, and the nature of the loading, whether dead or live. If live, then whether the application of the load is accompanied by severe dynamic shocks and pounding of the structure; whether the assumed loading for calculation is the absolute maximum, rarely to be applied in practice, or a possibility that may frequently take place. Prolonged heavy, steady loading, and also alternate tensile and compressive stresses in the same place, will call for lower averages. Information as to whether the assumed break-

ing stresses are based on full-size or small-size tests or only on interpolated values, averaged from tests of similar species of timber, is valuable in order to attribute the proper degree of importance to recommended average values. The class of timber to be used and its condition and quality. Finally, the particular kind of strain the stick is to be subjected to and its position in the structure with regard to its importance and the possible damage that might be caused by its failure.

"In order to present something definite on this subject, your committee presents the accompanying table, showing the average safe allowable working unit stresses for the principal bridge and trestle timbers, prepared to meet the average conditions existing in railroad timber structures, the units being based upon the ultimate breaking unit stresses recommended by your committee and the following factors of safety, viz:

Tension with and across grain.	10
Compression with grain.	5
Compression across grain.	4
Transverse rupture, extreme fiber stress.	6
Transverse rupture, modulus of elasticity.	2
Shearing with and across grain.	4

"In conclusion, your committee desires to emphasize the importance and great value to the railroad companies of the country of the experimental work on the strength of American timbers being conducted by the Forestry Division of the United States Department of Agriculture, and to suggest that the American Association of Railway Superintendents of Bridges and Buildings indorse this view by official action and lend its aid in every way possible to encourage the vigorous continuance of this series of Government tests, which bids fair to become the most reliable and useful work on the subject of strength of American timbers ever undertaken. With additional and reliable information on this subject far-reaching economies in the designing of timber structures can be introduced, resulting not only in a great pecuniary saving to the railroad companies, but also offering a partial

check to the enormous consumption of timber and the gradual diminution of our structural timber supply."

A very complete series of tests were made by Frank W. Hibbs, Naval Constructor, U. S. N., at the Puget Sound Navy Yard,

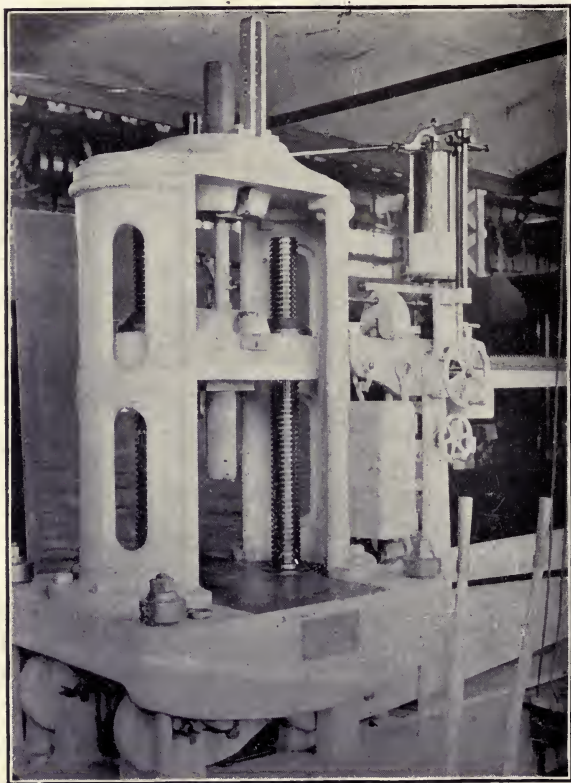


FIG. 137.—TENSILE TEST, DOUGLAS FIR.

on the "Comparative Tests of Yellow Pine and Puget Sound Fir." This is fully published in a paper of the Pacific Northwest Society of Engineers, to which the reader is referred, as it is impossible to quote even results here. Fig. 137 shows the method used for making the tensile tests, and Fig. 138 the method used for making the transverse tests. The conclusions drawn from these tests were as follows:

1st. Strength. Douglas fir is generally equal to yellow pine, and superior to it in some essential particulars.

2d. Elasticity. Douglas fir is decidedly more elastic than yellow pine.

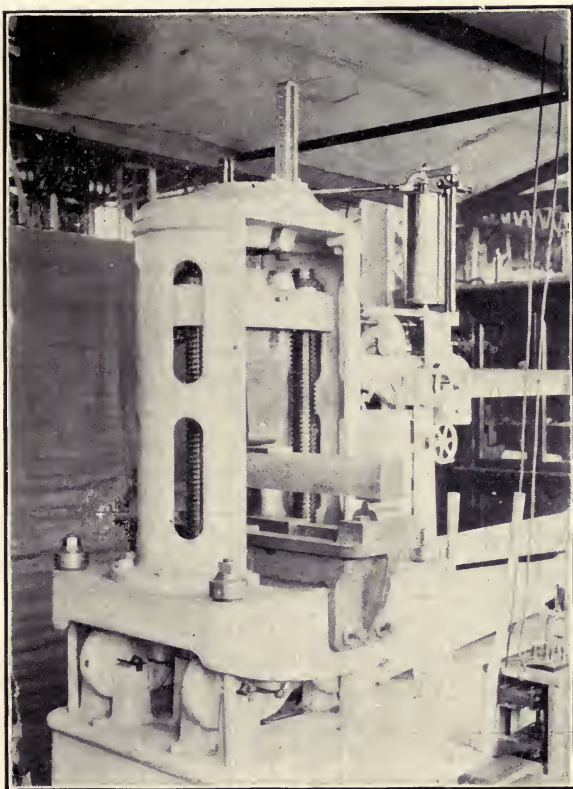


FIG. 138.—TRANSVERSE TEST DOUGLAS FIR.

3d. Toughness. Douglas fir is far superior to yellow pine as regards toughness.

4th. Wearing qualities. Yellow pine is superior to Douglas fir, especially when moisture is present.

5th. Lasting qualities. Yellow pine is superior to Douglas fir, on account of the greater amount of pitch that it contains.

TABLE IX.—AVERAGE ULTIMATE BREAKING UNIT STRESSES IN POUNDS PER SQUARE INCH RECOMMENDED BY THE COMMITTEE ON "STRENGTH OF BRIDGE TRESTLE TIMBERS," AMERICAN ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS, FIFTH ANNUAL CONVENTION, NEW ORLEANS, OCTOBER, 1895.

Kind of Timber.	Tension.		Compression.			Transverse Rupture.		Shearing.	
	With Grain.	Across Grain.	With Grain.		Across Grain.	Extreme Fiber Stress.	Modulus of Elasticity.		
			End Bearing.	Columns under 15 Diameters.					
White oak.	10,000	2,000	7,000	4,500	2,000	6,000	1,100,000	800	4,000
White pine.	7,000	500	5,500	3,500	800	4,000	1,100,000	400	2,000
Southern, longleaf, or Georgia yellow pine.	12,000	600	8,000	5,000	1,400	7,000	1,700,000	600	5,000
Douglas, Oregon, and Washington fir or pine: Yellow fir.	12,000		8,000	6,000	1,200	6,500	1,400,000	600	
Red fir.	10,000					5,000			
Northern or shortleaf yellow pine.	9,000	500	6,000	4,000	1,000	6,000	1,200,000	400	4,000
Red pine.	9,000	500	6,000	4,000	800	5,000	1,200,000		
Norway pine.	8,000		6,000	4,000	800	4,000	1,200,000		
Canadian (Ottawa) white pine.	10,000			5,000				350	
Canadian (Ontario) red pine.	10,000			5,000		5,000	1,400,000	400	
Spruce and Eastern fir.	8,000	500	6,000	4,000	700	4,000	1,200,000	400	3,000
Hemlock.	6,000			4,000	600	3,500	900,000	350	2,500
Cypress.	6,000		6,000	4,000	700	5,000	900,000		
Cedar.	8,000		6,000	4,000	700	5,000	700,000		1,500
Chestnut.	9,000			5,000	900	5,000	1,000,000	600	1,500
California redwood.	7,000			4,000	800	4,500	700,000	400	
California spruce.				4,000		5,000	1,200,000		

TABLE X.—AVERAGE SAFE ALLOWABLE WORKING UNIT STRESSES IN POUNDS PER SQUARE INCH RECOMMENDED BY THE COMMITTEE ON "STRENGTH OF BRIDGE AND TRESTLE TIMBERS," AMERICAN ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS, FIFTH ANNUAL CONVENTION, NEW ORLEANS, OCTOBER, 1895.

Kind of Timber.	Tension.		Compression.			Transverse Rupture.		Shearing.	
	With Grain.	Across Grain.	With Grain.		Across Grain.	Extreme Fiber Stress.	Modulus of Elasticity.	With Grain.	Across Grain.
			End Bearing.	Columns under 15 Diameters.					
Factor of safety.	10	10	5	5	4	6	2	4	4
White oak.	1,000	200	1,400	900	500	1,000	550,000	200	1,000
White pine.	700	50	1,100	700	200	700	500,000	100	500
Southern, longleaf, or Georgia yellow pine.	1,200	60	1,600	1,000	350	1,200	850,000	150	1,250
Douglas, Oregon, and Washington fir or pine:									
Yellow fir.	1,200	...	1,600	1,200	300	1,100	700,000	150	
Red fir.	1,000	...				800			
Northern or shortleaf yellow pine.	900	50	1,200	800	250	1,000	600,000	100	1,000
Red pine.	900	50	1,200	800	200	800	600,000		
Norway pine.	800	...	1,200	800	200	700	600,000		
Canadian (Ottawa) white pine.	1,000	...		1,000	...			100	
Canadian (Ontario) red pine.	1,000	...		1,000	...	800	700,000	100	
Spruce and Eastern fir.	800	50	1,200	800	200	700	600,000	100	750
Hemlock.	600	...		800	150	600	450,000	100	600
Cypress.	600	...	1,200	800	200	800	450,000		
Cedar.	800	...	1,200	800	200	800	350,000	...	400
Chestnut.	900	...		1,000	250	800	500,000	150	400
California redwood.	700	...		800	200	750	350,000	100	
California spruce.		...		800	...	800	600,000		

6th. Weight. Douglas fir is 14 per cent. lighter than yellow pine.

The main reason why timber is not used in many cases is on account of its short life. Upon exposure to the elements it decays quite rapidly, lasting on the average about ten years in structures, and when used for railroad ties only about seven years. If the surface is protected from the rain by painting this closes up the pores and causes heart rot from the moisture that is retained inside the stick of timber. For this reason paint should seldom be used upon a timber structure of any kind. Where timber is placed in salt water it is destroyed from other and quite different causes. The marine animals which are most destructive are the limnora, which eats off the surface of the timber near the water line, and the teredo, which eats out the interior. Untreated peeled piling driven in salt water become so badly eaten by the teredo in two or three months as to break off under almost no load at all, if this load is applied transversely. The view, Fig. 139, is of a section of piling which was eaten off in less than two years, it having been driven with the bark on. The bark is a good protection, and some of the methods of protecting piling against the teredo are based upon the plan of providing piles with an artificial bark formed of burlap, treated with asphaltum and wrapped about the piles with wires. The best protection, however, is creosoting, or the impregnating of piling or timber with creosote or the dead oil of coal tar.

The amount of creosote used varies from 10 to 20 pounds per cubic foot, from 12 to 10 pounds being the ordinary amount specified. For the preservation of timber simply against rot, 10 or 12 pounds is sufficient, while many engineers think it necessary to use from 16 to 20 pounds for protection against the teredo. Soft timber takes creosote readily, but some of the harder kinds, or those with hard fibers, will not take up the creosote so easily, so that it would seem advisable to specify the amount of penetration the creosote should have from the surface of the wood inwards instead of specifying the amount per cubic foot.

A penetration of five-eighths of an inch forms a satisfactory protection against the teredo.

As the other methods of preserving timber are less used, they

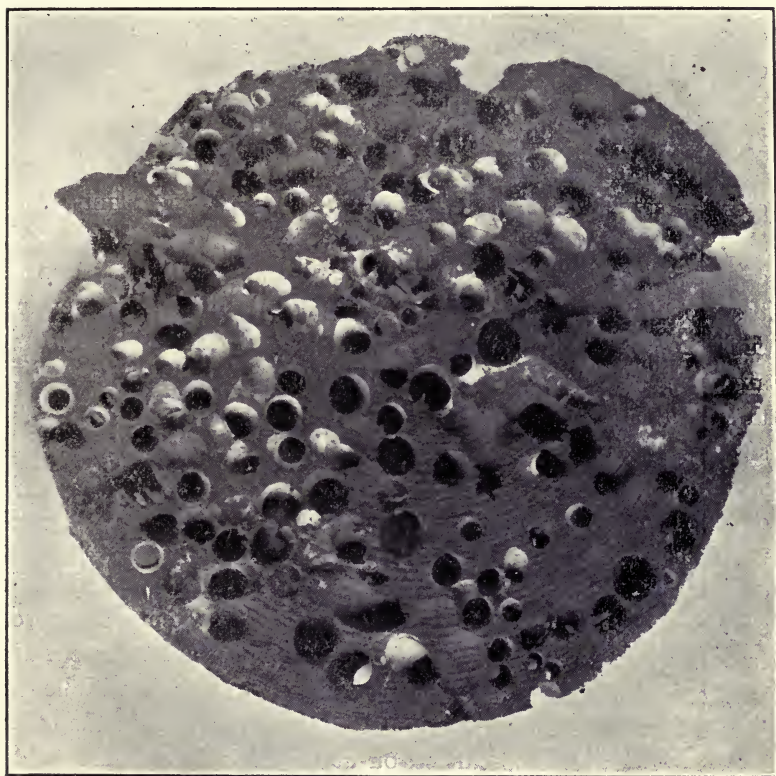


FIG. 139.—SECTION OF TEREDO-EATEN PILE.

will only be mentioned, and in case the engineer should find a plant available for preserving timber by one of the other methods its processes can readily be compared with those of creosoting. These methods are Kyanizing, or bichloride of mercury process; Burnettizing, or zinc chloride process; and Margaryizing, or sulphate of copper process. None of these, however, are so good as creosoting, inasmuch as the preserving material dissolves out of the timber and leaves it to decay. This feature is very strongly

brought out in some experiments that have been made with Egyptian mummies, as when all of the embalming material has been extracted, the mummies at once decay rapidly.

Creosoting plants are found in almost every section of the country, and a description of the methods followed at any one of these works practically covers the methods employed at others.

The description given by the Norfolk Creosoting Company is quoted here in full:

“The preservation of timber by the dead oil of coal-tar process, as carried on by all well-equipped creosoting plants, consists of two distinct operations—the preparation of the wood, and its impregnation with the preservative. The preparation of the wood necessary for the proper reception of the preserving substances is the removal of all those portions of the tissue which are subject to fermentative action. This consists of the extraction of the liquids and semi-liquids occupying the interfibrous spaces, and constituting the very immature portions of the wood, without softening the cement binding of the fibrillæ, or bundles of cellulose tissue, forming the solid or fully matured part. Upon the successful accomplishment of this entirely depends the value of artificially preserved wood for structural purposes. If this step of the operation is conducted at too low a temperature, or for too short a time, the sap or liquid part nearest the surface will only be extracted, the consequence of which will be an insufficient space for receiving the preservative. If, on the other hand, the operation is carried on at too high a temperature, or for too long a time, the resinous portion of the bundles of fibrillæ will be softened and the wood lose its elasticity in just the proportion that the coherence of the fibrillæ is lessened. The temperature should never be less than 100° C. or exceeding 130° C. Of the two possible methods for the removal of the undesirable portions of the timber, exposure to currents of dry air, and steaming under pressure with an after-drying in a vacuum, the latter is now the universal practice. While the first-named plan may seem the more rational, and the one least likely to modify injuriously the physical structure, such is not the case. Under proper

manipulation, a more thorough desiccation, without harmful change of the organic structure, can be accomplished in twelve hours less by the latter process, than is ever possible with air drying which, under the most favorable circumstances, is a long-drawn-out operation, and cannot do more than extract the water from that portion of the sap which has not yet reached the semi-solid stage, thus leaving in the tissues of the wood a very considerable amount of resinous matter which occupies space that should be ready to receive the creosote-oil. The consequence of this is a failure of the oil to reach many of the interfibrous passages, which are either left empty or are filled with the gelatinous part of the half-matured growth cells in which are to be found the conditions that make putrefaction possible. In order to remove the sap from wood, it is first necessary to vaporize it and then to bring about such external circumstances which shall allow outflow of all gaseous matter from the interior of the wood. In order to vaporize the sap it is necessary to break down the walls of the cells containing the liquid and semi-liquid substances. This is readily accomplished through the agency of heat applied through the medium of a moist steam-bath, at such a pressure as to keep the temperature of the wood, and its surrounding atmosphere, somewhat above the boiling-point of the sap. The maintenance of this condition for a few hours is found to be quite sufficient to break down the sap-cell tissue and to vaporize all those constituents that it is desirable to withdraw. This point having been reached, the steam-bath is discontinued; and the temperature being maintained at, or slightly above, the vaporizing-point of the sap, the pressure of the atmosphere surrounding the wood within the chamber is reduced below that of the interior of the wood. The result of this condition is an outflow of vapor and air, continuing until equilibrium is restored. This equilibrium is prevented by the use of an exhaust pump until the absence of aqueous vapor in the discharge from the pump indicates the completion of the operation. At this stage the wood-tissue is in a state very like that of a sponge cleared of hot water; every pore is gaping open and ready to receive the oil.

“ In the practice of the Norfolk Creosoting Company the most carefully dried lumber is steamed and subjected to the action of the heated ‘vacuum’ in order that there may be had that thorough and uniform penetration of the preserving liquid that is essential to the highest efficiency of the product. The timber having been thus prepared the creosote-oil is admitted to the chamber, which is still kept under the influence of the vacuum pump, at a temperature somewhat above the boiling-point of the sap,† at the pressure then existing in the chamber. As the hot oil envelops the wood and enters the interfibrous spaces, the aqueous vapor yet remaining in the wood, by reason of its less specific gravity, rises to the top of the containing chamber and is withdrawn by the pump. By the time that the chamber is entirely filled with oil, all the remaining moisture has escaped. The exhaust pump is stopped and, in order to facilitate the absorption of the oil by the wood, a pressure pump is set to work supplying oil to the chamber at such pressure as may be desired. This operation is continued until the requisite amount of oil has been put into the timber. The chamber is then opened and the timber withdrawn. The apparatus is then ready for further use.

“ The successful conduct of the operation above outlined exacts the most careful attention and skillful management, supplemented by adequate and suitable appliances. The wide divergence in the characteristics of timber; the varying amounts of sap, due to the lapse of time since, and the season in which the tree was felled; its possible subsequent immersion in water for a longer or shorter time; the character of the soil and the conditions under which the tree grew, whether in a dense forest or a comparatively open country, whether it is of a rapid, even growth, or a slow intermittent one, are all factors contributing to a more or less perfect product. To the experienced operator these conditions indicate, in each case, the proper course to be pursued. Failure to observe and to take them into consideration is to invite indifferent, uncertain and in the end unsatisfactory results. Of equal importance is a proper understanding of the circumstances under which the finished product is to be used. Timber for piers,

wharves and other structures in tropical waters demand processes and degrees of thoroughness of treatment that are unneces-

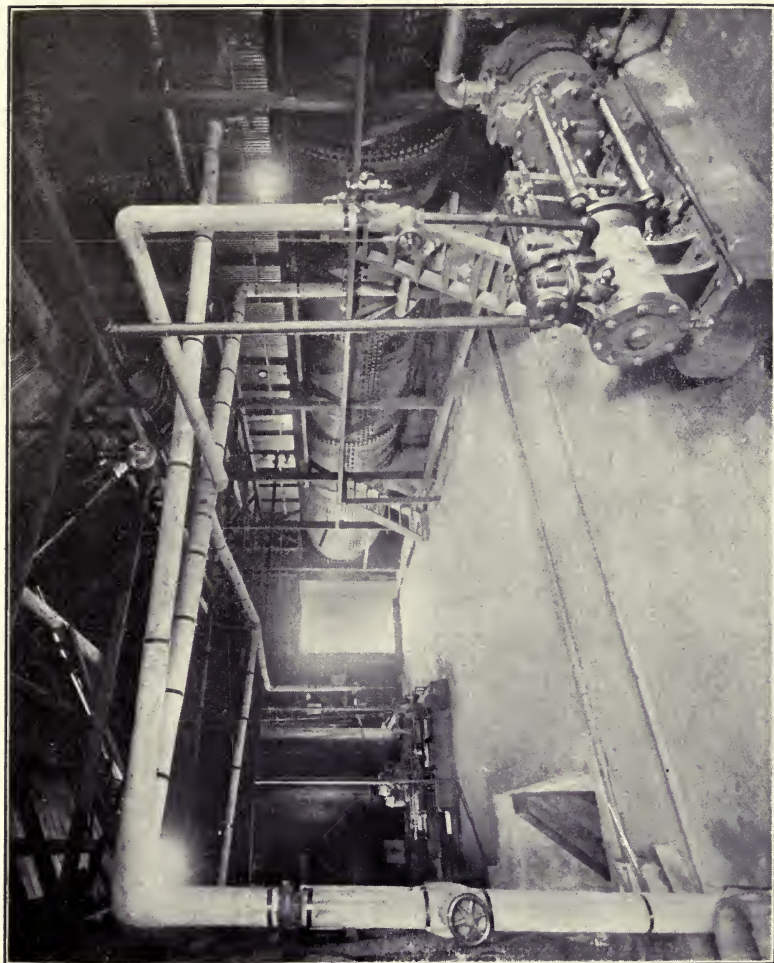


FIG. 140.—PLANT FOR CREOSOTING TIMBER.

sary in the harbors of more temperate climates, which are, in turn, more exacting than land and fresh-water construction.

“It is as true as it is unfortunate, that, in the past—perhaps at present—much creosoted work has fallen far below the reasonable expectation of the purchaser and user. As creosoting is

neither a secret or patented process, nor are its operations complex, a close and systematic inspection of materials used at the

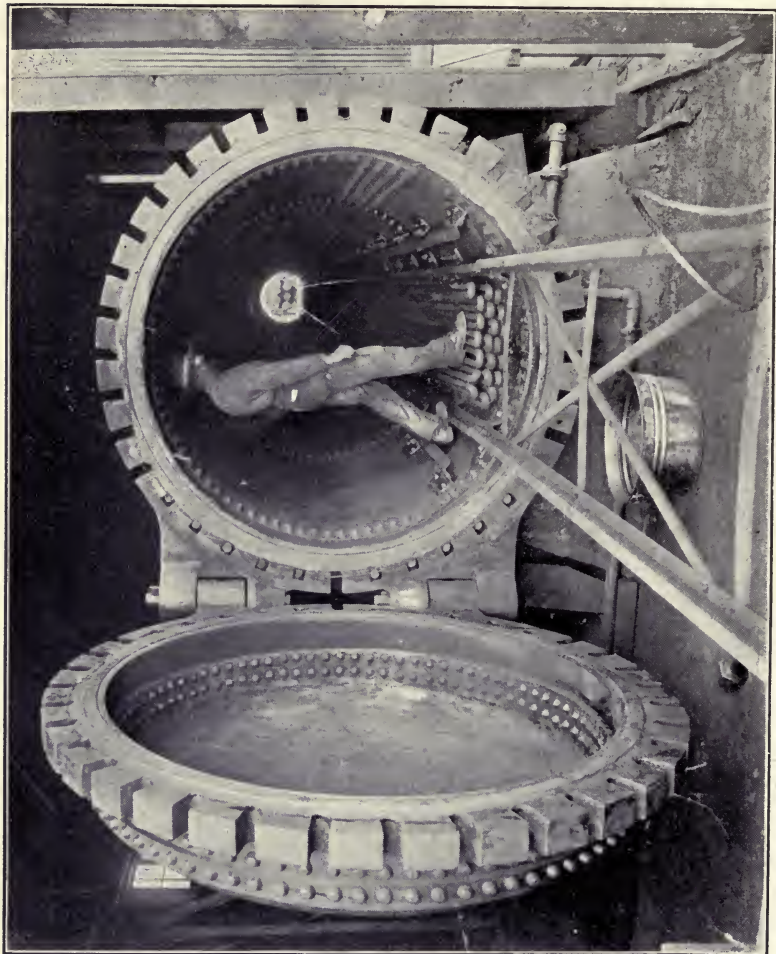


FIG. 141.—CREOSOTING RETORTS.

place of manufacture is all that is necessary for the buyer, and at the time that the creosoting is in progress.”

The cost of creosoting varies of course with the size of piling and the size of timbers and with the location as well, but as a general average the cost of treating ordinary sized piling with 10

pounds of creosote per cubic foot will add about twenty-five cents per lineal foot to the price of the piling, and the cost of creosoted trestle timbers will range from twenty-five dollars to thirty-five dollars per thousand feet B.M. in addition to the price of the timber. Views in a modern creosoting plant are shown in Figs. 140 and 141, which plant is located at Eagle Harbor, Washington. The plant is owned by the Perfection Pile Preserving Co. of Seattle, and the retorts are built strong enough to make it possible to fill specifications requiring large percentages of creosote to be put in fir timber, which is much harder to treat properly than other kinds.



Frank S. Lee

APPENDICES.

SELECTIONS FROM SPECIFICATIONS.

APPENDIX I.

SPECIFICATIONS FOR COFFER-DAMS AND FOUNDATIONS, OHIO RIVER MOVABLE DAMS.

MAJOR W. H. HEUER, U. S. Engineer.

GENERAL DESCRIPTION.

The site of Dam No. 2 is on the Ohio River, distant from Pittsburg, Pa., $10\frac{1}{2}$ miles, and adjacent on the right bank to the Pittsburg, Ft. Wayne and Chicago Railway. It has Neville Island on the left bank, and is accessible by street cars from Pittsburg.

The lock is to be located on the left bank of the Ohio River, immediately behind Merriman's dyke. It will be in general dimensions the same as locks Nos. 1 and 6, viz., 110 feet wide and 600 feet long.

SPECIAL DESCRIPTIONS.

The river bed at No. 2 consists of gravel throughout, and the excavations will be made to a depth sufficient to insure a permanent and enduring foundation, which will ordinarily be 14 feet below the gate sill, but may be otherwise, as the engineer, in his judgment, may direct.

The work will conform to the drawings exhibited, and to such others, in explanation of details or modifications of plans, as may be furnished from time to time during construction.

CONTRACTOR TO FURNISH ALL MATERIAL AND WORK.—It is understood and agreed that the contractor, under his contract prices for work in place, is to furnish and pay for all materials, stone, cement, sand, earth, timber, material for coffer-dam and protection cribs, excavation, lock-filling and

discharging valves (set in masonry), flushing valves, anchor bolts, lock-gate tracks, and everything entering into or connected with either the permanent or temporary construction, and he is also to supply and pay for all work, skilled and otherwise, required to prepare and place the materials, and complete the work according to the drawings and these specifications.

CONTRACT TO INCLUDE.—The contract will cover the construction and completion of the foundation, masonry and timber work of the lock, including both land and river walls, the gate-recess walls, the foundations of the lock-gate tracks, the guiding walls above and below the lock, the pipe and flushing conduits, the drift chute, the foundations for the power-house and lock-keepers' residence, and every such other permanent construction as shall be shown upon the drawings. It shall also include the clearing of the land necessary for the proper execution of the work embraced in this contract, all pumping and bailing, dredging and excavation, puddling and embankment, the construction of all coffer-dams, stone masonry, concrete and brick masonry, timber work and iron work, and all such other work which, in the judgment of the Engineer, is necessary and included in the proper completion of the contract.

TOOLS, MACHINERY, BUILDINGS, ETC.—The contractor, without cost to the United States, shall furnish all appliances, dredges, pumps and pumping machinery, boats, tools, derricks, tramways, foot-walks, roads and landings, and all needful temporary buildings and shops.

COFFER-DAMS.

SHEETING.—The coffer-dam, about 1,500 feet in length, shall be built as shown generally by the drawings exhibited, and as directed by the Engineer. It shall be 14 feet high above the sill of the lock, and shall consist of two walls or rows of plank sheeting, spaced 12 feet apart in the clear, driven or set firmly from 1 to 2 feet into the river-bed, and supported laterally by horizontal longitudinal stringers, the latter being spaced at varying intervals, increasing in width from the bottom to the top, and to be sufficiently and firmly bolted together transversely with iron rods passing through the coffer-dam horizontally from the rows of stringers on the one side to the corresponding rows on the other, against which the vertical plank sheeting shall be securely spiked.

FILLING AND DECKING.—The interior, or space between the walls of sheeting, shall be filled with heavy dredged river-bed or other material not liable to wash, and to be covered over with a suitable decking of plank (to protect it from injury in case of being submerged by floods), all complete as shown on the drawings.

PILING AND CRIBS TO PROTECT.—At the upper outer corner of the coffer-dam shall be placed a crib built of framing timber and filled with riprap stone; from the upper corner of the crib, at an angle of 45° with the axis of the current, a line of piling, spaced 5 feet apart, firmly bolted together

with waling-pieces, shall be driven to the shore to form a protection to the coffer-dam; also outside and along the coffer-dam, from the upper outer corner to the lower corner, clusters of piles, firmly bound or bolted together, shall be driven at intervals of about 80 feet. The tops of all piling shall be sawed off to a uniform height of 2 feet above the coffer-dam. Protection cribs shall be placed at such other points along the coffer-dam as may be shown upon drawings.

HOW PAID FOR.—Bidders will state a price per lineal foot of coffer-dam completed. No payment will be made for any portion thereof until the entire coffer-dam is completed. Drawings will be furnished, showing the general type of the coffer-dam and its manner of construction, and every detail necessary for intelligent bidding. Should any work on the outside of the coffer be necessary, such as gravel filling or riprapping, it shall be paid for at the price bid for gravel filling, riprapping, etc. If, owing to the nature of the river bed, it shall be found impossible to drive the plank sheeting to the required depth, then the contractor, after driving the sheeting as deep as possible without injury, and in lieu of driving it to its full depth, shall fill around the outside of the walls with the same material as is used in filling the coffer-dam, to the height of 4 feet above the surface of the river-bed, and for which no extra compensation will be allowed.

REMOVAL OF.—The contractor will be required to remove the coffer-dam and its belongings at his own cost. The time and manner of the removal of the coffer-dam, or any part thereof, and the place to deposit the materials, shall be prescribed by the Engineer.

TO BELONG TO THE UNITED STATES.—It is understood and agreed that the payments made for the coffer-dam, including the crib and pile protection, shall cover the entire cost thereof to the United States, and by virtue thereof they shall become the property of the United States. The contractor, however, must maintain the same and make all needed repairs to same during the existence of the contract, without expense to the United States.

DEPOSIT WITHIN THE COFFER-DAM.—Material washed or left in the space inclosed by the coffer dam by freshets shall be removed by the contractor, as directed, at his price for common excavation, which price shall cover all necessary cleaning and scrubbing. No payment will be made, however, for removing material washed into the inclosure from the coffer-dam itself or from any deposit made by the contractor on or above the works.

MATERIAL AND WORKMANSHIP.

TEMPORARY PILING shall include all piles driven for the protection of the coffer-dam and "deadmen" for derricks. They shall be of good quality, round oak timber, not less than 12 inches diameter at the butt, and of length varying from 20 to 25 feet, and longer if necessary.

SHEET PILING.—In excavating for foundations, should quicksand or

fine sand carrying water be encountered, close sheet-piling will be required to be driven to whatever extent the Engineer may direct.

SHEETING.—The sheeting shall include the walls and decking of the coffer-dam, including the stringers; also such shoring as may be directed by the Engineer to remain in the finished structure. It shall consist of the best quality of hemlock obtainable, and must be in all cases satisfactory to the Engineer in charge.

GRAVEL OR EARTH FILLING.—Gravel or earth filling will include all material used in filling the land-wall inclosure, back of the guiding walls, etc. It does not include any filling in the construction of the coffer-dam.

STONE FILLING shall include all stone placed in the protection cribs or any riprap stone ordered for the protection of the work.

CRIBWORK shall be built of hemlock framing timber framed together in square bins and securely bolted together by iron drift-bolts. The interior of the cribs shall be filled with riprap stone, and should the Engineer deem it necessary such riprap stone shall be placed on the outside of the crib. The whole to be built as shown by the drawings.

FRAMING TIMBER.—For all temporary cribwork, also the permanent crib at the head of the upper guiding wall, framing timber shall be used. No stick shall be less than 10" $\times$ 10" in section.

"Framing timber" is a commercial term for a class of timber hewn to various sizes.

EXCAVATION.

TO INCLUDE.—It shall include the removal of all gravel or other material to the depth required for the lock and its upper and lower entrances, the gate recesses, Poiree-dam and gate-track foundations, for the foundations of all walls, and for all conduits or wells, and all such other material as may be found necessary in the judgment of the Engineer to be removed for foundations and otherwise in permanent construction. It will include all dredging and all material excavated of whatever nature, however removed, for foundations and for site of coffer-dam.

LINES, SLOPES, AND GRADES FOR.—All excavations shall conform to such lines, slopes, and grades as may be given by the Engineer, and anything taken out beyond such given limits will not be paid for by the United States.

MATERIAL TO BE DEPOSITED.—Excavated material is to be deposited as and where directed by the Engineer. It shall be deposited in such manner as not to interfere with present or proposed navigation. Material of any kind deposited by the contractor in absence of, or in disregard of, instructions, shall, if required by the Engineer, be removed by the contractor at his own cost.

SHORING.—All excavation for foundation shall be securely shored and thus maintained until the foundation has been sufficiently advanced to dispense with the same, when it may remain or be removed at the discretion of the Engineer.

DREDGES AND PUMPS.—The contractor will be required to employ, at the same time, not less than two suitable steam dredges at excavating and filling; and for pumping he must keep at least three good sufficient pumping outfits, with pumps, engines, and boats complete, in or always ready for operation. The dredges must be equipped to do effective work to a depth of 28 feet.

FOUNDATIONS.

CHANGES OR MODIFICATIONS OF.—The character of the river-bed and of the proposed foundations for the different parts of the work is shown in general on the drawings and cross-sections exhibited, and it is understood that the United States shall have the power to make any changes in the plans of the foundations that may, in the judgment of the Engineer, be considered advisable after examinations made, as the excavations proceed within the coffer-dam after it is pumped out, and it is understood and agreed that the contractor shall have or make no claim against the United States on account of any such changes in or modifications of the plans of the foundations, or on account of any increase or decrease in the depth of same, under or over those referred to herein or shown on the drawings exhibited.

MASONRY.

CEMENT.—Cement will be of uniform quality, setting well both in air and water, and free from anything that will cause the mortar to swell, crack, or scale. It shall be put up in strong, sound barrels, well lined with paper so as to be reasonably protected from air and moisture. The average net weight of the barrels shall be not less than 265 pounds, unless expressly so stated in the proposal. Each barrel must be labeled with the name of the brand and of the manufacturer.

In general, ten barrels of every one hundred will be tested.

The cement must stand the following tests: Fineness—At least 85 per cent. must pass a sieve of 6,400 meshes to the inch. Setting—Cement must be moderately slow setting; it must not begin to set within fifteen minutes, as determined by Vicat needle $\frac{1}{12}$ inch in diameter with $\frac{1}{4}$ pound load, and it shall not bear the weight of one pound on wire $\frac{1}{24}$ inch in diameter within thirty minutes, but must bear such weight within one hour and a half. Strength—The minimum tensile strength per square inch of briquettes of neat cement mixed with about 33 per cent. of water by weight, and exposed in air for one hour, and the remainder of 24 hours in water, shall be not less than 50 pounds; with longer time, whether in air or water, there must be a decided increase of strength; it must also test to the satisfaction of the Engineer when mixed with sand. The tests for setting will be made at a temperature of air and water of about 75° Fahrenheit. All other tests will be made at a temperature above 60° Fahrenheit. The cement will be subject to inspection at all times, and must be kept well housed.

SAND.—The sand used must be clean, sharp, washed, river sand, satisfactory to the Engineer.

MORTAR.—To be composed generally of two parts of sand to one of cement; when required, and whenever thought necessary by the Engineer, it shall be made richer. It must be thoroughly mixed and used before it has begun to set. If required by the Engineer, the mortar beds will be protected from the sun.

POINTING.—All face work is to be pointed, as fast as the work progresses, with stiff mortar, mixed one of sand to one of Portland cement, thoroughly hammered in and finished with proper tools; before the final acceptance of the work all face masonry which at that time does not appear properly pointed shall be repointed by the contractor to the satisfaction of the Engineer, without extra cost.

FROST.—Masonry will not be executed during freezing weather, nor when, in the judgment of the Engineer or his agent, it is likely to freeze before the mortar shall set. To guard against injury from frost, all new and unfinished work shall be properly protected by the contractor at his own cost.

VOIDS AND OPENINGS.—Due regard shall be had in the construction of all masonry walls to leave all necessary voids or openings for conduits or wells, or for such other purposes as may be required by the Engineer.

ASHLAR.—It shall comprise such part of the walls as is built of stone, with point-dressed face, and beds, and joints smoothly and squarely dressed.

QUALITY OF STONE.—All stone shall be perfectly sound, strong, hard, free from injurious seams, and in all respects satisfactory to the Engineer. Stone to be such as can be truly wrought to such lines and surfaces, whether curved or plain, as may be required. No stone shall be used which weighs less than 135 pounds to the cubic foot.

SAMPLES OF STONES.—Each bidder must deposit at this office, all charges prepaid, before the bids are opened, a 6-inch cubical block of the stone he proposes to furnish, and state the quarry from which it was obtained. The quality of the stone must be at least equal to that of the sample. The sample must be truly squared, and dressed on four sides; one side must be hammer-dressed, one side smooth-dressed and rubbed, and one side pitch-dressed. The other two sides are to be left with quarry face.

STONE MAY BE REJECTED.—The United States reserves the right to reject any stone not deemed suitable, or which, during the execution of the contract, shall be found defective. The beds of the stone must be their natural quarry beds. No lewis or dog holes, letters, or marks of any kind will be allowed on any dressed face of stone, but each face shall have left on it a boss for lifting, to be removed by the contractor after the stone has been set.

DRESSING OF STONE.—Stone must be accurately cut, square, and true, and the faces must be pitch-draughted and point-dressed to a plane with the draught, forming an approximately smooth surface. The beds must be

smoothly and squarely dressed, full length and width. The vertical joints must be dressed to a depth of not less than 18 inches from the face, and the allowance for joints must not exceed $\frac{3}{8}$ inch. One-third of the stone in each course must be headers. All stones not accurately dressed will be rejected. All dressed stone must have the dimensions plainly marked on one end.

DIMENSIONS.—The cut-stone stretchers must be not less than 3 feet nor more than 5 feet long, and their width must be not less than $1\frac{1}{3}$ times the height of the course to which they belong. The width of the headers must be not less than $1\frac{1}{3}$ times their height, and their length must be at least double their breadth, unless otherwise ordered. The thickness of course includes the joint, which will be $\frac{3}{8}$ inch.

LAYING STONE MASONRY.—The faces of the walls shall be accurately laid to the lines indicated on the drawings, or as directed by the Engineer. All stones to be well laid to proper lines, in full beds of mortar, and settled in place with a wooden maul; the use of grout is prohibited. No dressing, except in special cases, and by permission of the Engineer, will be allowed on backing after it is laid in the wall. The bond of stone shall in no case be less than 9 inches. The walls will be laid in horizontal courses throughout, each course to be of uniform height through the wall. Heights and arrangements of courses to be determined by the Engineer. When laying masonry the site for the stone shall be thoroughly cleaned with a scrub-broom and moistened; and the stone shall always be cleaned and well moistened before being set. Not more than three unfinished courses of face stone will be permitted upon the wall at the same time without special permission from the Engineer in each case. Proper machinery must be used in handling the stone; face stone shall not be disfigured by use of plug or grabs. Any stone chipped or spalled shall be rejected. Stones having defects concealed by cement or otherwise will be rejected on that account alone.

COPING.—The coping will be of the same class and quality of stone described in ashlar masonry. It will be carefully and truly cut to forms and dimensions given, from the best stone; it will be crandalled on all outer faces; the exposed edges of the coping to be rounded to a radius of 3 inches and chiseled smooth where required. Beds and vertical joints to be pointed true and full throughout and be laid with $\frac{3}{8}$ -inch joints.

The coping is to be doweled as required by the Engineer with round iron. The dowels to be furnished and placed by the contractor. The drilling for and placing of the dowels will be covered by the price for "Bolt Holes in Masonry." The dowels will be set in Portland cement.

RUBBLE STONE.

QUALITY AND DIMENSIONS OF.—Rubble stone must be sound, hard, and durable, free from seams, scale, earthy matter, and other defects. Rubble stone shall in general be not less than $\frac{3}{4}$ of a cubic foot in size. It must be

in fair shape for laying in the face of the walls without dressing. No spalls will be allowed.

LAYING.—The stone must be laid on their natural bed in full beds of hydraulic cement mortar, with all joints and voids well filled with mortar. Leveling up under stones with small chips or spalls will not be allowed.

The stone shall be carefully selected for the outer face so as to have vertical joints and present a good face of broken rough masonry.

CONCRETE.

COMPOSITION OF.—Concrete shall be composed of satisfactory cement and river gravel; the latter, should it be of an approved quality, shall be taken from the various excavations of the lock and its walls. This gravel generally has a sufficient volume of sand to fill all voids; should there be a deficiency of sand in any portion of the gravel the contractor will be required to supply said deficiency by good, sharp, washed, river sand. The quantity of cement to be used will generally be about 20 per cent. greater than the volume of voids in sand and gravel.

MIXING AND PLACING OF.—The concrete is to be well and rapidly mixed by machinery, as may be required by the Engineer, unless otherwise specified. It will be deposited in layers not more than 8 inches thick; wherever and whenever required, the layers will be thinner than 8 inches, and all thoroughly rammed by such process as the Engineer may approve.

RIVER WALL.—In the river wall of the lock the concrete shall be laid in courses of a thickness corresponding to the adjoining courses of ashlar masonry. It shall be filled in flush with the top of each course before the next course of ashlar above shall be laid.

Before putting in the concrete of any course the bed and adjoining course of ashlar shall be thoroughly wetted so that no dry surface may come in contact with the fresh concrete, destroying its power of adhesion by absorbing its moisture.

In order that the work once begun may progress without delay all cut stone needed for the ashlar facing shall be on the ground when the concrete foundation has been completed.

TIMBER IN PERMANENT CONSTRUCTION.

TO CONSIST OF all timber used in the timber facing of the lock walls and the guide walls; all timber cribbing in the gate-track and Poiree-dam foundations; the oak sheeting at the head of the guide walls; and such other timber in permanent construction as shall be shown upon the drawings.

GENERAL QUALITY AND DIMENSIONS.—All timber must be first class, and any of inferior quality will be rejected. Sap-wood in any stick will cause its rejection. The timber must be free from black or rotten knots,

wane edges, wind-shakes, dose, or other imperfections. Firm, sound knots, if not too numerous, will not be considered defects. Timber must be full size, true, and out of wind, and when required must be sawed large enough to dress down to required dimensions. The timber will be inspected on arrival at the work, and if found to be defective will be rejected.

OAK.—Oak timber must be taken from the best quality live white oak sawed timber.

WHITE PINE.—Shall consist of the best quality of clear white pine obtainable.

HEMLOCK.—Shall be the best quality of hemlock obtainable.

FRAMING, ASSEMBLING, AND PAINTING.—All timber must be accurately framed, fitted, and assembled, according to detailed drawings and directions. As the timber is framed it shall be painted about the ends and elsewhere as may be required to prevent checking. The paints for this will be furnished and applied by the contractor, and covered in his price for "Timber in Permanent Construction."

TIMBER FACING, UPRIGHTS, AND SHEETING shall be constructed of oak, and shall consist of uprights spaced at intervals of 6 feet, center to center, anchored to the concrete masonry by tee-head screw bolts as shown on drawings. To the uprights shall be bolted, with wrought-iron screw-bolts, oak sheeting 6 inches thick.

NOSING TIMBER shall extend along the top of the guide wall, forming a cap to the uprights and securely bolted to them, as shown on the drawings. The top surface of the nosing shall be flush with the top of the concrete masonry wall.

OAK SHEETING.—This refers to the sheeting on the upper faces of the protection crib for the upper guiding wall at the upper end thereof. It shall be spiked on and firmly held in place with iron bands or straps bolted to the framing timbers of the crib, if, in the judgment of the Engineer, this may be deemed necessary.

SUPERVISION AND MEASUREMENT OF WORK.

INSPECTION, REJECTED MATERIAL, ETC.—The works will be conducted under the direction of the local or resident Engineer, who shall have power to prescribe the order and manner of executing the same in all its parts; of inspecting and rejecting materials, work, and workmanship which, in his judgment, do not conform to the drawings that may be furnished from time to time, or to these specifications. And any material, work, or workmanship so rejected by him shall be kept out of or removed from the finished work, and no estimate or payment shall be made until such material, work, or workmanship be so removed.

When so required rejected material shall be piled up in sight near the works and kept there until the Engineer gives permission to have it removed.

The United States will keep inspectors on the work who will receive

instructions from the resident Engineer. They will have power to object to any materials, work, or workmanship. Any material, work, or workmanship objected to by the inspectors shall be kept out of or removed from the finished work, unless in each particular case the objections of the inspector shall be overruled by the local or resident Engineer; and, unless the objection be so overruled, no estimate or payment shall be made until such material, work, or workmanship be so removed.

The local or resident Engineer shall have power to overrule or rescind any or all objections or decisions of the inspector.

The decision of the United States Engineer Officer in charge of the works shall be final and conclusive upon all matters relating to the work and upon all questions arising out of these specifications, and from his decision there shall be no appeal.

FAILURE TO PROSECUTE OR PROTECT WORKS.—If at any time the contractor shall refuse or fail to prosecute the work or provide for carrying on the same as directed by the Engineer, or fail to properly protect any part of the work, permanent or temporary, the Engineer shall have power to employ men, to purchase or otherwise provide materials, tools, machinery, etc., and put the work in proper advancement or condition, and the entire cost of so doing shall be deducted from payments to be made under this contract.

COMPLETE WORK REQUIRED.—The contractor is not to take advantage of any omissions of details in drawings or specifications, or errors in either, but he will be required to do everything which may be necessary to carry out the contract in good faith, which contemplates everything complete, in good working order, of good material, with accurate workmanship, skillfully fitted and properly connected and put together. Any point not clearly understood is to be referred to the Engineer for decision.

CHANGES.—Should any changes in the details of the shape, arrangement, or fitting of the parts be deemed necessary or advisable in the progress of the work, they must be made by the contractor, and a fair allowance will be paid for any changes which, in the judgment of the Engineer in charge, materially increases the cost of the work.

MEASUREMENT.—Measurement of all work and material shall be made in place, unless otherwise specified.

COFFER-DAM.—The price per lineal foot of coffer-dam shall include all material, lumber, iron, and gravel entering into its construction. A profile of the location will be furnished, showing a section of the river-bed over which the coffer-dam is located, so that the contractor may estimate the amount of each kind of material required.

PILING.—Temporary piling shall be measured in lineal feet, and measurement shall be allowed for total length of piling used.

SHEETING.—This will include all lumber used for temporary purposes, in shoring of excavations, or for forms necessary to sustain any concrete

masonry until it has become sufficiently hardened. Sheeting required by the Engineer to remain in the finished structure shall be paid for at the contractor's price per thousand feet B.M. All temporary sheeting not remaining in the finished structure shall be included in the contractor's unit price for material in place, and no estimate will be made thereof by the Engineer. Cofferdam sheeting will be included in the contractor's price per lineal foot of coffer-dam.

FILLING.—Gravel filling will be measured in the fill, and will not include any filling placed in the coffer-dam as coffer-dam filling.

Stone filling shall include all riprap work, either temporary or permanent.

EXCAVATION.—Excavation will be measured in excavation by cross-sections.

MASONRY.—All masonry, ashlar, rubble, brick, concrete, etc., will be measured by the cubic yard in place. Prices for masonry will include all required pointing. No payment will be allowed for voids or openings.

BOLT HOLES.—All holes drilled in rock or concrete or other masonry will be measured by the running foot as drilled.

TIMBER IN PERMANENT CONSTRUCTION.—Timber in permanent construction will include all timber used in any part of the permanent construction; unless otherwise particularly specified, it will be classed under the following heads:

Oak in Permanent Construction.

Pine in Permanent Construction.

Hemlock in Permanent Construction.

APPENDIX II.

EXTRACTS FROM TOPEKA (KANSAS) MELAN ARCH BRIDGE SPECIFICATIONS.

By permission of EDWIN THACHER, M. Am. Soc. C. E.

PILING IN PERMANENT WORK.

Piling and lumber for coffer-dams to be sound white oak, yellow pine, or other woods equally good for the purpose, the quality to be acceptable to the superintendent. The piles shall be straight-grained, trimmed close, and have all bark taken off, and shall be at least 10 inches in diameter at the small end and 14 inches in diameter at the butt when sawed off. The heads shall be cut off squarely at right angles to the axis of the pile, and all piles shall be fitted to and driven with a cast-iron head. The piles shall be driven with a hammer weighing not less than two thousand two hundred and fifty (2250) pounds, and until they do not move more than three-eighths ($\frac{3}{8}$) of an inch under a blow of the hammer falling twenty-five (25) feet. No pile shall be driven less than twenty-six (26) feet below low water, and if necessary to attain this minimum depth jets shall be used in addition to hammer. The number and arrangement of the piles for each foundation are shown on the plans, and must be carefully carried out by the contractor. The piles shall be cut off at an elevation of about six (6) inches below low water. A slight variation will be allowed, but no piles must be cut off at a higher elevation. Inspection of piling and lumber, except at bridge site, shall be at contractor's expense.

COFFER-DAMS.

After the bearing piles have been driven, a permanent coffer-dam, of the dimensions marked on the plans, of Wakefield (or other equally satisfactory) sheet piling, shall be used around each foundation. The earth inside thereof shall be excavated to the depth shown on plans and replaced with concrete as hereinafter specified. During the placing of the concrete the water shall be kept out of the coffer-dams, unless the bottom is so porous that it is impracticable in the opinion of the superintendent to do so—in

which case some of the concrete may be placed in position by means of chutes, under the direction of the superintendent until the bottom is well calked, after which the water shall be pumped out and the remaining concrete placed in position. The contractor will be required to make the sides and ends of the coffer-dams watertight, and no leak through them will be considered sufficient cause to require any concrete to be placed by means of chutes.

CENTERING.

The contractor shall build an unyielding falsework, or centering, of the form and dimensions shown on the plans; particular care must be taken to drive the piles supporting it to a solid bearing. The estimated load upon each of these piles is twenty (20) tons. The contractor must, however, satisfy himself as to the load each pile will have to bear, and as to its supporting power. In case of any settlement the contractor shall take down and rebuild the centering and arch. The lagging shall be dressed on both edges to a uniform size, so that when laid it will present a smooth surface, and this surface shall be built at the proper elevation to allow for settlement of arch, so that when the centering is struck the arch-ring will come to the elevations shown on plans.

The top surface of the lagging shall be covered with W. Field's Building Paper of medium weight, known as Double Saturated Water-proof Oiled Sheathing Paper (or other equally good) to prevent the concrete from adhering thereto. No center shall be struck until at least twenty-eight (28) days after the completion of the arch. Great care shall be used in lowering the centers so as not to throw undue strains upon the arches, nor shall any center be struck before the adjoining arch has been completed for a sufficiently long time, in the opinion of the superintendent, to be uninjured thereby.

NOTE.—For the above reasons it is probable that the five centers will be in use at the same time.

PORTLAND CEMENT.

The Portland cement shall be a true Portland cement, made by calcining a proper mixture of calcareous and clayey earths; and the contractor shall furnish one or more certified statements of the chemical composition of the cement and of the raw materials from which it is manufactured. Only one brand of Portland cement shall be used on the work, except with permission of the superintendent, and it shall in no case contain more than two (2) per cent. of magnesia in any form.

The fineness of the cement shall be such that at least 98 per cent. shall pass through a standard brass cloth sieve of 74 meshes per linear inch, and at least 95 per cent. shall pass through a sieve of 100 meshes per linear inch.

Samples for testing may be taken from each and every barrel delivered, as superintendent may direct. Tensile tests will be made on specimens

prepared and maintained, until tested, at a temperature of not less than 60° Fahrenheit. Each specimen shall have an area of one square inch at the breaking section, and after being allowed to harden in moist air for twenty-four hours shall be immersed and retained under water until tested.

The sand used in preparing the test specimens shall be clean, sharp, crushed quartz, retained on a sieve of 30 meshes per linear inch and passed through a sieve of 20 meshes per linear inch, and shall be furnished by contractor.

No more than 23 to 27 per cent. of water by weight shall be used in preparing the test specimens of neat cement, and in making the test specimens one of cement to three of sand, no more than 11 or 12 per cent. of water by weight shall be used.

Specimens prepared from neat cement shall after seven days develop a tensile strength not less than 400 pounds per square inch. Specimens prepared from a mixture of one part cement and three parts sand (parts by weight) shall after seven days develop a tensile strength of not less than 140 pounds per square inch, and after twenty-eight days not less than 200 pounds per square inch. Specimens prepared from a mixture of one part cement and three parts sand (parts by weight) and immersed, after twenty-four hours, in water to be maintained at 176° Fahrenheit, shall not swell nor crack, and shall after seven days develop a tensile strength of not less than 140 pounds per square inch.

Cement mixed neat with about 27 per cent. of water, to form a stiff paste, shall, after 30 minutes, be appreciably indented by the end of a wire one-twelfth inch in diameter, loaded to weigh one-quarter pound.

Cement made into thin cakes on glass plates shall not crack, scale, or warp under the following treatment: Three pats shall be made and allowed to harden in moist air at from 60° to 70° Fahrenheit; one of these shall be subjected to water-vapor at 176° Fahrenheit for three hours, after which it shall be immersed in hot water for forty-eight hours; another shall be placed in water at from 60° to 70° Fahrenheit, and the third shall be left in moist air.

Samples of one-to-two mortar and of concrete shall be made and tested from time to time as directed by the superintendent. All cement shall be housed and kept dry till wanted in the work.

Storage rooms and rooms and apparatus for the tests shall be furnished by the contractor, and all tests shall be made entirely at his expense, and under the direction and to the satisfaction of the superintendent.

PORTLAND CEMENT CONCRETE.

The concrete shall be composed of clean, hard, broken limestone (or gravel with irregular surfaces) and cement mortar in volumes as hereinafter described. The sand shall be clean, sharp, Kansas River sand, washed *entirely* free from earth and loam. If obtainable, a mixture of coarse and

fine sand shall be used. Approved mixing machines shall be used. These machines must be kept clean and no accumulations of old mortar shall be allowed to form in them. The ingredients shall be placed in the machine in a dry state and in the volumes specified and be thoroughly mixed, after which clean water shall be added and the mixing continued until the wet mixture is thorough and the mass uniform. No more water shall be used than the concrete will bear without quaking in ramming. The mixing must be done as rapidly as possible, and the batch deposited in the work without delay, and before the cement begins to set. Stone must be entirely free from earth and earthy surfaces. Thin splints or leaves of stone, easily broken with fingers, will not be allowed to go into the work. The quality of stone and the crushing must be acceptable to the superintendent.

The grades of concrete to be used are as follows (parts by volume):

For the arches: One part Portland cement, two parts sand, and four parts broken stone (hazelnut size, from one-half inch to one inch), except for the exposed faces and soffits of the arches, which shall have at least one inch in thickness of mortar composed of one part Portland cement and two parts sand.

For the piers, abutments, spandrel and wing-walls: On the exposed surfaces for at least one inch thick, one part Portland cement and two parts sand; for the next seven (7) inches, one part Portland cement, two parts sand, and four parts broken stone of hazelnut size. For the remaining portions: One part Portland cement, four parts sand, and eight parts broken stone of size to pass through a 3-inch ring, except such portions of the interior of the piers and abutments as are above the top of the cornice, or elevation 15.75 feet above low water, which shall be composed of one part Portland cement, three parts sand, and six parts broken stone which will pass through a 2½-inch ring.

No plastering of surfaces will be allowed nor any practice that will develop planes or surfaces of demarcation other than those hereinafter described. Immediately after the removal of any forms or centers, sand and cement shall be sifted on the surfaces and the surfaces rubbed hard with a float as may be directed by the superintendent.

During warm and dry weather and whenever the superintendent shall direct, all newly built concrete shall be kept well shaded from the sun and well sprinkled with water at the surface for several days or until well set.

There must be no definite plane or surface of demarcation between the facing and the concrete backing. The facing and the backing must be deposited in the same layer and well rammed in place at the same time.

In connecting old concrete with new, in the planes hereafter described, the old concrete shall be cleaned and roughened and soaked with water, and at the points of contact a mortar composed of one part cement and two parts sand shall be used and shall be laid in the same manner as specified for laying the facing.

NATURAL CEMENT CONCRETE.

The concrete around piles, to take the place of the earth excavated from the coffer-dams, shall be composed of one part natural cement, equal to the best Fort Scott, Kas., cement, three parts sand, and six parts of broken stone of the size to pass through a 3-inch ring. This concrete may be mixed by hand on platforms adjoining the foundations and shoveled directly into the coffer-dams, care being taken to deposit it in uniform layers of about 6 inches each and to carefully ram each layer.

PIERS, ABUTMENTS, AND SPANDRELS.

All piers, abutments, spandrels, and wing-walls shall be built in timber forms. These forms shall be substantial and unyielding, of proper dimensions for the work intended and closely jointed, and all surfaces that come in contact with the concrete shall be smoothly dressed and well oiled with linseed oil to prevent the concrete from adhering to them. That portion next to the exposed faces of the work need not be oiled, but shall be covered with oiled paper, the same as that specified for the centers.

Molds, to form molding and panels, smoothly finished and well oiled with linseed oil, shall be properly placed in the forms so that the finished work will appear as shown on the plans. Extreme care must be used to place them in proper position before placing any concrete or mortar in them.

CONTINUOUS WORK.

The following divisions shall constitute sections for continuous work, viz.: Each footing course of piers or abutments; each pier or abutment from footing course to cornice; each pier or abutment from cornice to springing line of arch; each spandrel wall from keystone to pier or abutment; each pier or abutment spandrel wall; that portion of the piers or abutments above springing line of arch shall be considered part of the longitudinal sections of the arch previously described.

Each of the above sections shall be carried on continuously night and day if necessary; that is, each layer shall be well rammed in place before the previously deposited layer shall have time to partially set.

Care shall be taken to make the joints (for expansion) in each spandrel wall over piers as indicated on the plans.

CONCRETE IN COFFER-DAMS.

The natural cement concrete in the coffer-dam shall extend from depths marked on plans to 1 foot below low water. Upon this concrete the footing courses of piers and abutments shall be founded.

The sheet-piling of coffer-dams shall be cut off at least down to low-water mark, neatly and evenly, by the contractor before the completion of the work.

APPENDIX III.

EXTRACTS FROM KATTE'S MASONRY SPECIFICATIONS.

By permission of WALTER KATTE, M. Am. Soc. C. E.

EXCAVATIONS will be classified under the following heads, viz.: Earth, hardpan, loose rock, solid rock, and excavation in water.

EARTH will include clay, sand, gravel, loam, decomposed rock and slate, stones and boulders containing less than one cubic foot, and all other matters of an earthy nature, however compact, excepting only "hardpan," as described below.

HARDPAN will consist of tough, indurated clay or cemented gravel which, in the opinion of the Engineer, requires blasting for its removal.

LOOSE ROCK.—All boulders and detached masses of rock measuring over one (1') cubic foot in bulk, and less than one (1) cubic yard; also all slate, shale, soft friable sandstone and soapstone, and all other materials excepting rock, solid ledge, and those described above; also stratified rock in layers of not exceeding eight (8") inches in thickness, when separated by strata of clay, and which, in the judgment of the Engineer, may be removed without blasting, although blasting may occasionally be resorted to.

SOLID ROCK will include all rock found in ledges, or masses of more than one (1) cubic yard, which, in the judgment of the Engineer, may be best removed by blasting, with the exception of stratified rocks described under the head of Loose Rock. In rock excavations the "bottom" must in all cases be taken down truly to sub-grade; and when so ordered by the Engineer ditches must be formed at the foot of the slope.

The contract price for excavations will apply to pits required for foundations of masonry when water is not encountered, and the price for

EXCAVATION IN WATER will only apply to foundation pits under water and deepening of channels in running water; it must cover all classes of material, and include drainage, bailing, pumping, and all materials and labor connected with such excavations, also the necessary dressing of the rock.

CEMENT must be of the best quality of freshly burned and ground hydraulic

cement, and be equal in quality to the best brands of cement. It will be subject to test made by the Engineer or his appointed inspector, and must stand a proof tensile test of fifty (50) pounds per square inch of sectional area on specimens allowed a set of thirty (30) minutes in air and twenty-four (24) hours under water.

MORTAR will in all cases be made of one part in bulk of the best hydraulic cement to two parts in bulk of clean, sharp sand, well and thoroughly mixed together in a clean box of boards, before the addition of the water, and must be used immediately after being mixed. No mortar left overnight will, under any pretext, be allowed to be used. The sand and cement used will at all times be subject to inspection, test, acceptance, or rejection by the Engineer.

CONCRETE.—Concrete shall be composed of fragments of hard, sound, and acceptable stone, broken to a size that will pass through a two (2") inch ring in any direction, thoroughly clean and free from mud, dust, dirt, or any earthy admixture whatever; mixed in the proportion of two (2) parts in bulk of the broken stone to one (1) part of fresh-made cement mortar of the quality herein described; and is to be quickly laid in sections and in layers not exceeding nine (9) inches in thickness, and to be thoroughly rammed until the mortar flushes to the surface; it shall be allowed at least twelve (12) hours to "set" before any work is laid on it.

FOUNDATIONS.

GENERAL DESCRIPTION.—Foundations for masonry shall be excavated to such depths as may be necessary to secure a solid bearing for the masonry, of which the Engineer shall be the judge. The materials excavated will be classified and paid for, as provided for in these specifications, under the general head of Excavations; and in case of foundations in rock, the rock must be excavated to such depth and in such form as may be required by the Engineer, and must be dressed level to receive the foundation course.

When a safe and solid foundation for masonry cannot be found at a reasonable depth (to be judged of by the Engineer), there will be prepared by the contractor such artificial foundations as the Engineer may direct. All materials taken from the excavations for foundations, if of proper quality, shall be deposited in the contiguous embankment; but any material unfit for such purpose shall be deposited outside the roadway, or in such place as the Engineer shall direct, and so that it shall not interfere with any drain or watercourse.

TIMBER.—Timber foundations when required shall be such as the Engineer may by drawings or otherwise prescribe, and will be paid for by the one thousand feet, board measure. The price, covering cost of material, framing, and putting in place, and all wrought- and cast-iron work ordered by the Engineer, will be paid for per pound, the price including cost of material, manufacture, and placing in the work.

PILING.—All timber used in foundations or foundation piling shall be of young, sound, and thrifty white oak, yellow pine, or other timber equally good for the purpose, acceptable to the Engineer. Piles must be at least eight (8") inches in diameter at the small end and twelve (12") inches in diameter at the butt when sawn off; they must be perfectly straight and be trimmed close, and have the bark stripped off before they are driven. They must be driven into hard bottom until they do not move more than one-half inch under the blow of a hammer weighing two thousand (2,000) pounds, falling twenty-five (25') feet at the last blow. They must be driven vertically and at the regular distances apart from centers, transversely and longitudinally as required by the plans or directions of the Engineer; they must be cut off squarely at the butt and be well sharpened to a point, and when necessary, in the opinion of the Engineer, shall be shod with iron and the heads bound with iron hoops, of such dimensions as he may direct, which will be paid for the same as other iron work used in foundations.

The necessary length of piles shall be ascertained by driving test piles in different parts of the localities in which they are to be used; and in case a pile shall not prove long enough to reach "hard bottom" it shall be sawed off square, and a hole two (2") inches in diameter be bored into its head twelve (12") inches deep; into this hole a circular white-oak trenail twenty-three (23") inches in length shall be well driven, and another pile similarly squared and bored, and of as large a diameter at the small end as can be procured, shall be placed upon the lower pile, brought to its proper position, and driven as before directed. All piles, when thus driven to the required depth, are to be cut off truly square and horizontal at the proper height given by the Engineer, and only the actual number of lineal feet of the piles left for use in the foundations after being sawn off will be paid for.

COFFER-DAMS.—Where coffer-dams are, in the opinion of the Engineer, required for foundations the prices provided in the contract for timber, piles, and iron in foundations will be allowed for the material and work on same, which is understood as covering all risks from high water or otherwise, draining, bailing, pumping, and all materials connected with the coffer-dams. Sheet-piling will be classed as plank in foundations, and will be paid for per one thousand (1,000') feet, board measure, if left in the ground.

TIMBER.

All timber must be sound, straight-grained, and free from sap, loose or rotten knots, wind-shakes, or any other defect that would impair its strength or durability; it must be sawed (or hewed) perfectly straight and to exact dimensions, with full corners and square edges; all framing must be done in a thoroughly workmanlike manner, and both material and workmanship will be subject to the inspection and acceptance of the Engineer.

APPENDIX IV.

SPECIFICATIONS FOR STEEL COFFER-DAM.

DESIGN.—The shell shall be made of elliptical shape for ordinary piers and circular for pivot piers. It shall be made not less than four feet larger than footing of pier in plan, to allow for variation in position during sinking.

The plates used shall be as large as can be handled with ease in the shop, during shipment, and during erection.

The splices may be either lap or butt joints, provided a good tight job will result, and the rivets must be spaced according to boiler-maker's rules.

The joint may be made tight by calking or by the use of a calking strip, but in either event the result must be guaranteed.

The shell must be stiffened by horizontal stiffening angles, girders, or trussing, to resist deformation during the placing and to resist both the quiescent and a maximum unbalanced earth or water pressure, or a wind pressure.

The bottom plates shall be re-enforced with narrow plates inside and outside, to form a wedge-shaped cutting edge; and when there is rock or hard bottom, the plates shall be cut to conform to its contour as nearly as possible.

The top shall be properly stiffened, and if necessary provided with connection holes for additional sections.

The factor for safety shall in no case be less than four, and in case the shell will be subject to shock, not less than six.

No metal of a less thickness than $\frac{1}{4}$ inch shall be used for temporary work, nor less than $\frac{3}{8}$ inch for permanent work in fresh water or $\frac{1}{2}$ inch in salt water.

MATERIAL.—The entire shell shall be constructed of the grade of steel known as soft medium, except rivets, which shall be of bridge quality of iron.

The steel may be made either by the Bessemer or open-hearth process, and the phosphorus shall never exceed 0.08 per cent.

Soft medium steel shall have an ultimate strength of from 55,000 to 65,000 pounds per square inch, as determined from standard test pieces; an elastic limit of not less than one-half the ultimate strength; an elongation of not less than 25 per cent. in 8 inches; and a reduction of area at fracture of not less than 50 per cent.

Samples to bend cold 180° to a diameter equal to the thickness of the sample, without crack or flaw on the outside of the bent portion.

ERECTION.—The erection must be done in a first-class manner, and all rivets must have full heads. The shell shall be placed in position within one-half the distance allowed for error in the design of the coffer-dam. Only a reasonable variation will be allowed for difference in level.

PAINTING.—All the metal work shall be thoroughly cleaned of rust or scale at the shops and coated thoroughly with hot asphaltum.

Before erection, in the field, it shall be given a second coating of hot asphaltum.

SEALING.—When in position on the bottom, if the coffer-dam has not been sunk through impervious strata, it shall be sealed by concreting around the circumference inside with concrete passed through a tube.

REMOVAL.—Should the coffer-dam not form a part of the permanent foundation it shall be taken apart, at the joints designed for the purpose, and carefully removed in such a manner as not to injure the foundation, and so as to be used again if required.

APPENDIX V.

SPECIFICATIONS FOR CEMENT SUBMITTED TO THE AMERICAN SOCIETY FOR TESTING MATERIALS.

Engineering News, June 30, 1904.

IN our report last week of the annual meeting of the American Society for Testing Materials we stated that the Committee on Cement had submitted a report containing a recommended specification for natural and Portland cements, which specification was approved by the meeting and will now be voted on by the society at large, through letter-ballot. This specification is herewith presented in full. Its importance arises mainly from the fact that this society aims to produce specifications which shall be, as nearly as possible, standards for this country. The committees of the society are carefully selected from the most prominent representatives of the interests concerned and are intended to give equal hearing to manufacturer and consumer. The Committee on Cement was composed of thirty-one members, as follows:

George F. Swain, Professor of Civil Engineering, Mass. Inst. of Technology, Boston, Mass. (Chairman); George S. Webster, Chief Engineer, Bureau of Surveys, Department of Public Works, Philadelphia (Vice-Chairman); Richard L. Humphrey, Consulting Engineer and Chemist, Harrison Building, Philadelphia, Pa. (Secretary); Pooth, Garrett & Blair, Engineers and Chemists, 406 Locust St., Philadelphia, Pa.; T. J. Brady, President, Coplay Cement Manufacturing Co., Coplay, Pa.; C. W. Boynton, Inspector of Cements, Baltimore & Ohio Railroad Co., Wheeling, W. Va.; Spencer Cosby, Major, Corps of Engineers, U. S. A., War Department, Washington, D. C.; Allan W. Dow, Inspector of Asphalts and Cement for the District of Columbia, Washington, D. C.; L. Henry Dumary, President, Helderberg Cement Co., Albany, N. Y.; A. V. Gerstell, General Manager, Alpha Portland Cement Co., Easton, Pa.; Edward H. Hagar, Manager, Cement Department, Illinois Steel Co., Chicago, Ill.; Wm. H. Harding, President, Bonneville Portland Cement, Land Title Bldg., Philadelphia; Lathbury & Spackman, Engineers and Chemists, 1619 Filbert St., Philadelphia; Robert W. Lesley, President, American Cement Co., 22 So. 15th St., Phila-

delphia, Pa.; F. H. Lewis, Manager, Virginia Portland Cement Co., Fordwick, Va.; John B. Lohar, President, Vulcanite Portland Cement, Land Title Bldg., Philadelphia, Pa.; Andreas Lundteigen, Assistant Manager, Peerless Portland Cement Co., Union City, Michigan; W. W. Maclay, President, Glens Falls Portland Cement Co., Glens Falls, N. Y.; Charles A. Matcham, Manager, Lehigh Portland Cement Co., Allentown, Pa.; Charles F. McKenna, Consulting Chemist, Laboratories, 221 Pearl St., New York; Spencer B. Newberry, Manager, Sandusky Portland Cement Co., Sandusky, Ohio; F. H. Bainbridge, Assistant Engineer of Bridges, Illinois Central Railroad Co., Chicago, Ill.; J. Madison Porter, Professor of Civil Engineering, Lafayette College, Easton, Pa.; Joseph T. Richards, Chief Engineer, Maintenance of Way, Pennsylvania Railroad Co., Broad St. Sta., Philadelphia, Pa.; Clifford Richardson, Director, New York Testing Laboratory, Long Island City, New York; Louis C. Sabin, Asst. Engineer, United States Engineer Office, Detroit, Michigan; Harry J. Seaman, Superintendent, Atlas Cement Co., Northampton, Pa.; S. S. Voorhees, Engineer of Tests, Supervising Architect's Office, Treasury Department, Washington, D. C.; Olaf Hoff, Engineer of Structures, New York Central & Hudson River Railroad, Grand Central Sta., New York; W. S. Eames, President, American Institute of Architects, St. Louis, Missouri; W. J. Kelly, Vice-President, American Railway Engineering and Maintenance of Way Association, Minneapolis, Minn.

The specifications drawn up by this committee are prefaced by some explanatory remarks and suggestions in regard to the various tests. For further explanations of the procedure of testing the committee refers to the report of the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, whose recommendations it adopts as standard and reprints as an appendix to its report. We have omitted this appendix and give below only the specifications and the prefatory remarks:

GENERAL OBSERVATIONS.

1. These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2. The committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

3. SPECIFIC GRAVITY.—Specific gravity is useful in detecting adulteration or underburning. The result of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

4. FINENESS.—The sieve should be kept thoroughly dry.

5. TIME OF SETTING.—Great care should be exercised to maintain the

test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities vitally affect the rate of setting.

6. **TENSILE STRENGTH.**—Each consumer must fix the minimum requirements for tensile strength to suit his own conditions. They shall, however, be within the limits stated.

7. **CONSTANCY OF VOLUME.**—The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

8. In making the pats the greatest care should be exercised to avoid initial strains due to molding or to too rapid drying out during the first twenty-four hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

9. The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement may, however, be held for twenty-eight days, and a retest made at the end of that period. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

STANDARD SPECIFICATIONS FOR CEMENT.

1. **GENERAL CONDITIONS.**—All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the contractor and a period of at least twelve days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 pounds of cement net. Each barrel of Portland cement shall contain four bags, and each barrel of natural cement shall contain three bags of the above net weight.

8. Cement failing to meet the seven-day requirements may be held awaiting the results of the twenty-eight-day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by

the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the society Jan. 21, 1903, and amended Jan. 20, 1904, with all subsequent amendments thereto.

10. The acceptance or rejection shall be based on the following requirements:

NATURAL CEMENT.

11. DEFINITION.—This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

12. SPECIFIC GRAVITY.—The specific gravity of the cement, thoroughly dried at 100° C., shall be not less than 2.8.

13. FINENESS.—It shall leave by weight a residue of not more than 10 per cent. on the No. 100, and 30 per cent. on the No. 200 sieve.

14. TIME OF SETTING.—It shall develop initial set in not less than ten minutes, and hard set in not less than thirty minutes, nor more than three hours.

15. TENSILE STRENGTH.—The minimum requirements for tensile strength for briquettes 1 inch square in cross-section shall be within the following limits, and shall show no retrogression in strength within the periods specified:

Neat cement:

Age.	Strength.
24 hours in moist air.	50-100 lbs.
7 days (1 day in air, 6 days in water).	100-200 "
28 " (1 " " " 27 " " ")	200-300 "
<i>One part cement, three parts standard sand:</i>	
7 days (1 day in air, 6 days in water).	25-75 "
28 " (1 " " " 27 " " ")	75-150 "

16. CONSTANCY OF VOLUME.—Pats of neat cement about 3 inches in diameter, one-half inch thick at center, tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(a) A pat is then kept in air at normal temperature.

(b) Another is kept in water maintained as near 70° F. as practicable.

17. These pats are observed at intervals for at least twenty-eight days, and, to satisfactorily pass the tests, should remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

PORTLAND CEMENT.

18. DEFINITION.—This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent. has been made subsequent to calcination.

19. SPECIFIC GRAVITY.—The specific gravity of the cement, thoroughly dried at 100° C., shall be not less than 3.10.

20. **FINENESS.**—It shall leave by weight a residue of not more than 8 per cent. on the No. 100, and not more than 25 per cent. on the No. 200 sieve.

21. **TIME OF SETTING.**—It shall develop initial set in not less than thirty minutes, but must develop hard set in not less than one hour, nor more than ten hours.

22. **TENSILE STRENGTH.**—The minimum requirements for tensile strength for briquettes one inch square in section shall be within the following limits, and shall show no retrogression in strength within the periods specified:

Neat cement:

Age.	Strength.
24 hours in moist air.....	150-200 lbs.
7 days (1 day in air, 6 days in water).....	450-550 "
28 " (1 " " " 27 " " ").....	550-650 "

One part cement, three parts sand:

7 days (1 day in moist air, 6 days in water).....	150-200 "
28 " (1 " " " " 27 " " ").....	200-300 "

23. **CONSTANCY OF VOLUME.**—Pats of neat cement about 3 inches in diameter, one-half inch thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(a) A pat is then kept in air at normal temperature and observed at intervals for at least twenty-eight days.

(b) Another pat is kept in water maintained as near 70° F. as practicable, and observed at intervals for at least twenty-eight days.

(c) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for five hours.

24. These parts, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, checking, cracking or disintegration.

25. **SULPHURIC ACID AND MAGNESIA.**—The cement shall not contain more than 1.75 per cent. of anhydrous sulphuric acid (SO_3), nor more than 4 per cent. of magnesia (MgO).

APPENDIX VI.

METAL SHEET-PILING.

THE prediction was made, in the first edition of this book, at the end of Chapter VI, that Metal Sheet-piling would doubtless come into use as timber became more expensive to use, and at the end of Chapter VIII mention is made of work at Cuxhaven Harbor, Germany, where Metal Sheet-piles were used. An account is also given of the Friestedt Patent Interlocking Sheet-piling, which is almost identical with Metal Sheet-piling described in Volume I of the "Transactions of the Institution of Civil Engineers." It is worthy of comment that this has probably been lost sight of by engineers, and even "The Engineer" in a review of the first edition of this book makes the statement:

"Numerous existing examples of coffer-dams constructed of sheet-piling are described and illustrated, and Mr. Fowler endorses a statement recently made in our columns by remarking that "the growing scarcity of timber will doubtless lead to the use of metal at some time in the future to replace sheet-piling for coffer-dams." So that the following paper on Metal Sheet-piling, published in 1836, will doubtless prove of great interest to engineers, if not of considerable value:

"Memoir on the use of Cast Iron in Piling, particularly at Brunswick Wharf, Blackwall. By Michael A. Borthwick, A. Inst. C. E."

A short sketch of the introduction and use of cast iron in piling may not be considered an inappropriate accompaniment to an account of one of the most recent works in which it has been adopted.

Public attention was first drawn to such an application of iron by Mr. Ewart, of Manchester, now of His Majesty's Dock-yard, at Woolwich; but though this merit is certainly due to that ingenious gentleman, he had been, as it afterwards proved, anticipated in the idea by the late Mr. Mathews, of Bridlington, who, previously to the date of Mr. Ewart's patent, had used cast-iron sheet-piles in the foundations of the head of the north pier of that harbor. These piles were of different forms; in the margin (Fig. 142) is given a cross-section of one of, I believe, the most common, in which it will be seen the adjoining piles dovetail to each other, while in others, I have been informed, they merely overlap. Their length was about 8 or 9 feet, their width from 21 inches to 2 feet, and their thickness half an inch.

In ignorance of Mr. Mathews' proceedings, Mr. Ewart, in the beginning of 1822, took out a patent for a new method of making coffer-dams, which he proposed to effect by employing plates of cast iron, held together by cramps fitted to dovetailed edges on the piles. A section of these piles, taken from some that have been used, is shown in the accompanying sketch (Fig. 143). A detail of the mode in which it was proposed to combine them so as to form a coffer-dam might be out of place, in a paper that has reference more to the use of iron piling for permanent purposes; the plan, as described in the specification of the patent, is to be found in the *Repertory of Arts*, and an abstract of it in the *London Journal of Arts and Sciences* for the year 1822. The length of the piles is therein stated as intended to be from 10 to 15 feet, which is, I understand, about what they have generally been made, and for



FIG. 142.—MATTHEW'S CAST-IRON SHEET-PILE.

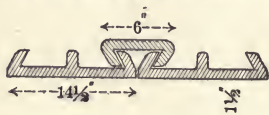


FIG. 143.—EWART'S CAST-IRON SHEET-PILE.

cases requiring a greater depth, a mode is described of lengthening the piles, by placing one above another, and securing the horizontal joints by means of dovetailed cramps.

Though, on being apprised of what had been done at Burlington, Mr. Ewart did not defend his patent, his piles have been pretty extensively adopted, particularly by Mr. Mylne, of New River Head, London, and Mr. Hartley, of Liverpool. Besides other operations in the important public work under his charge, the former gentleman used the piles, soon after their invention, with complete success in a coffer-dam of considerable size, constructed in the river Thames for the purpose of putting in a suction-pipe opposite the New River Company's establishment at Broken Wharf. They have also been used with advantage by Mr. Hartley, in founding the pier heads of the basin of George's Dock, and various parts of the walls of some of the other docks at Liverpool, as also in putting in the foundations of the south river-wall.

Looking at the dovetailed form of these piles, one would, I think, have been inclined to anticipate difficulty in driving them, but this does not seem to have been met with to any extent in practice, at least in coffer-dams, the original object of the invention. On this point I have pleasure in being able to quote some observations of Mr. John B. Hartley, which contain the results of the Liverpool experience: "Considerable care," he writes, "is required in keeping the piles in a vertical position, as they are apt to shrink every blow and drive slanting. They require to be driven between two heavy barks of timber to keep them in a straight line, as they expose very little section to the blow of the ram, and are so sharp that they are easily driven out of a right line. There is another very necessary precaution to

be taken, which is the keeping of the fall in the same line as the pile; otherwise the ram descending on the pile and not striking it fairly, all parts equally, the chances are that, if in a pretty stiff stratum, the head breaks off in shivers, and the pile must be drawn, which is sometimes no easy matter." He concludes by saying, "these piles are on the whole the most useful tools you can use for their purpose (coffer-damming). I believe they have had as extensive a trial at the Liverpool Docks as anywhere else, and certainly with success. They have generally been driven with the ringing or hand engine and rams of 3 or 4 cwt., a front and back pile being driven at the same time by one ram."

In the work at Broken Wharf, the practice was to insert the piles and cramps all round the dam first, and drive them a moderate distance into the ground, then to pass the engine repeatedly round and send them down gradually, instead of driving them home at once; and Mr. Mylne has mentioned to me that while this was in progress, the piles being at the time but slightly driven, he was somewhat alarmed one morning at finding that the run of the water had elevated one end of the dam considerably above the other. The dovetails, however, held good, and proper precautions being taken, the return of the tide put all right again without at all crippling the work, the movement having been regular all over the dam. I ought to add that these dams are still used in the works on the New River, four sets being generally kept in hand, and that the ringing engine is always employed, and the above stated method of driving followed.

I have perhaps dwelt longer on Mr. Ewart's project than I should otherwise have done, from a feeling that from his labors has sprung much that has followed in the way of iron piling; and besides, it may be observed, the remarks as to driving are not entirely limited in their application to this particular description of pile. The next work that occurs was executed by Mr. Walker in 1824; this was the rebuilding of the return end of the quay-wall of Downes Wharf, Saint Katherine's, which had been undermined by the wash from the Hermitage entrance of the London Docks. With a view to a more effectual resistance of a like action in future, iron instead of wood sheet-piling was introduced in the foundation of the wall in question; and though, if one may judge from the specification of the patent, no application of his plan of so permanent a nature seems to have been contemplated by Mr. Ewart, the work was begun according to it, but it was afterwards modified at the request of the contractor, so as to give the section of pile shown in the margin (Fig. 144), the flanch being in front or outside.

Although, as has been already seen, the piles in their original form may be easily enough driven in some cases, it was found impossible to get them down in a regular line to the depth required in the present instance, through the hard material

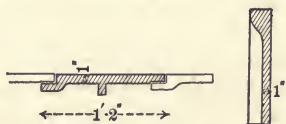


FIG. 144.—EWART'S MODIFIED SHEET-PILE.

that had to be penetrated, and by which in fact they were surrounded and pressed for nearly their whole length of 14 feet.

A work on a much larger scale than any yet mentioned now presents itself, the wharfing at the sea entrance of the Norwich and Lowestoft navigation.

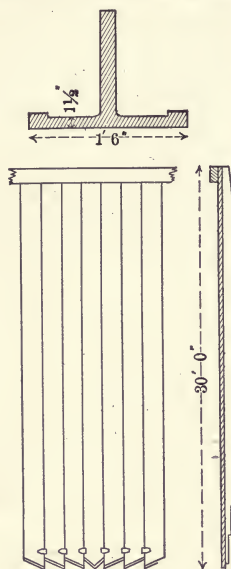


FIG. 145. — CUBITT'S IRON SHEET-PILING.

In this Mr. Cubitt has adopted sheet-piling, exclusively without the intervention of main or guide piles; the form and section will be seen by the accompanying sketches (Fig. 145), which it is almost unnecessary to observe are not drawn to the same scale, the transverse section being considerably enlarged beyond the other two. The piles are all 30 feet long; their weight is about a ton and a half each. The back flanch, which is shown at the deepest on the cross-section, tapers gradually to about 6 inches at top, where the angles are blocked in to form a head for driving, and is diminished at the lower end by steps or set-offs of parallel width with square ends, instead of a straight or curving line, as the latter shape was found to have a tendency to press the pile forward, whereas by the plan adopted it drove as fairly as if the flanch had been continued its full width to the foot of the pile. The driving was all effected by means of crab engines with monkeys about as heavy as the piles, no more fall being allowed than was necessary to send them down, and the whole is secured by land ties, two in height, at intervals of six feet. The entire length of wharfing thus constructed is about 2,000 feet.

From the form of the pile, according to this plan, giving so thin an abutting surface, and the joints not being covered in any way, close and accurate driving seems essential to its efficacy, and the nature of the ground (sand mixed with shingle) would have made this a somewhat troublesome operation at Lowestoft, but for the plan that was taken to insure precision. This consisted in riveting close to the lower end of the pile about to be driven a pair of strong wrought-iron cheeks projecting beyond the edge about 2 or 3 inches, which clasping the pile already driven, served as a guide or groove to keep the piles flush, however this the edge * and the tendency to turn out or in at the heel was counteracted after a few trials by giving a greater or less bevel to the front or back face. With these appliances the piling was pretty closely driven, and the work, which was completed in 1832, has been

* This plan has, I believe, been followed by Mr. Cubitt in driving timber-piling also, in cases requiring nicety of work.

found fully to answer the object of supporting the sides of the cut from Lake Lothing to the sea against the effects of the very ingenious and powerful sluicing apparatus provided in the lock at that place.

About a year later than the above, Mr. Sibley constructed an iron wharfing on the Lea Cut at Limehouse on an opposite principle, sheet-piling being in this case altogether discarded, and the work consisting of flat plates let down in grooves on the sides of guide-piles of an elliptical form according to the section opposite (Fig. 146), driven at distances of 10 feet. These piles are 20 feet long, weigh about $1\frac{1}{4}$ tons each, and are 9 feet apart; they are hollow throughout, to enable a passage for them to be bored in the soil by means of an auger passed through them, and so ease the driving and are filled with concrete; each pile is land-tied, and the plates extend to within 6 feet of the point. A similar wharfing, but on a larger scale, has since been made on each side of the Thames, adjoining New London Bridge; that on the city side rather an extensive work, the piles in it being 43 feet long (cast in two unequal lengths with a spigot and faucet joint), of a cylindrical form, 12 inches diameter, and of metal $1\frac{1}{2}$ inches thick, and each pile being secured by two tiers of ties of 2-inch-square iron carried 70 or 80 feet back, to resist the great depth of filling up or backing.

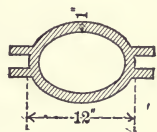


FIG. 146.—SIBLEY
HOLLOW IRON
SHEET-PILE.

The plan just described seems well enough adapted for situations where any great increase of depth is not likely to take place. The absolute depth is not so important, though where this is considerable, it may be questionable whether a heavy wharf would not be the better for the protection of a continuous row of piling at foot; the strong land-tying necessary in the last-mentioned work seems to point to this.

I now come to the quay-wall constructed in 1833-34 by Messrs. Walker and Burges on the river Thames, in front of the East India Docks at Blackwall, and since named Brunswick Wharf. The object of this work was to afford accommodation for the largest class of steam-vessels at all times of tide, for which the old quay, even had it not been in a state of decay, was not adapted from the shallowness of the water in front of it. To effect this, the first idea was to run out two or three jetties from the wharf, but this was soon abandoned, and a new river-wall resolved on; and advantage was taken of the occasion to improve the line of frontage by an extension into the river, under the sanction of the Navigation Committee of the Port of London, varying from a point at the east end to about 25 feet at the other extremity. The use of iron in the work was, I have understood, suggested by Mr. Cotton, deputy chairman at the time, and for many years an active member of the most respectable and liberal body then in the direction of the East India Dock Company, and the adoption of the proposal was facilitated by the circumstance which probably led in the first instance to its

being made, namely, the low price of the material at the period, the contract being little more than 7 pounds per ton delivered in the Thames.

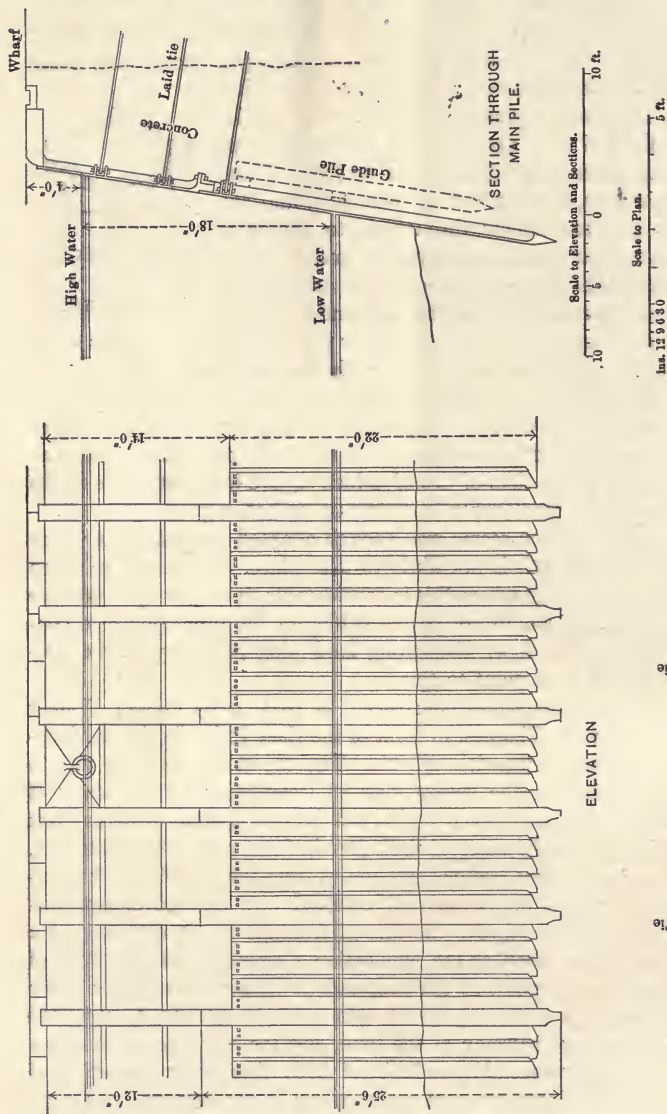


FIG. 147.—BRUNSWICK WHARF IRON SHEET-PILING.

In the accompanying drawing (Fig. 147) an attempt is made to show the mode of construction that was followed, so as to avoid the necessity for

much written detail. The first operation was to dig a trench 2 yards deep in the intended line, and this was immediately followed by the driving of the timber guide-piles. The deepening in front, which, to give the required depth of 10 feet at low water, was as much as 12 feet, was not done until near the conclusion of the work; to have effected it in the first instance would without any countervailing advantage, except some saving in the driving, have been attended with the double expense of removing the ground forming the original bottom between the old and new lines of wharfing, and afterwards refilling the void so left by a material that would require time to make it of equal solidity; and even if this had been otherwise, such an attempt would have endangered the old wall, or rather would have been fatal to it. The permanent piling was next begun, the main piles being driven first at intervals of 7 feet, and the intermediate spaces or bays then filled in, working always from right to left, towards which the drafts of the sheet-piles were pointed. The ground is a coarse gravel, with a stratum of the hard Blackwall rock occurring in places, and some trouble was occasionally experienced from its tendency to turn the piles from the proper direction, but, due attention being paid to the form of the points, the driving was on the whole effected pretty regularly, but few of the bays requiring closing piles specially made for them, so that the work may be said to be nearly iron and iron from end to end; at the same time, the vertical joints of the piling being all covered, as will be noticed presently, any slight imperfection in this respect is no, serious detriment to the work as a whole.

The main piles are in two pieces, the lower end of the upper one being formed so as to fit into a socket on the top of the under length, and the joining made good by means of a strong screw-bolt; the only object of this was to insure a supply of truer castings, and lessen the difficulty of transporting such unwieldy masses from Northumberland and Staffordshire to London.* Each sheet-pile is secured at the top by two bolts to the uppermost wale of the woodwork behind, and the edge of the end ones of each bay, it will be observed, passes behind the adjoining main pile, while the other joints are overlapped by the bosses with which all the sheet-piles except the closers are furnished on one side. Besides adding to the perfection and security of the work by breaking the joints, so that the water (if it penetrate, as with even the best pile-driving it will) cannot draw the backing from its place, these projections appear to me to relieve the appearance of the otherwise too uniform face; and a like effect is produced by the horizontal fillets on the lower edges of the plates above, which also mask the joints. These plates, filling up the spaces over the sheet-piling, are bolted to the main piles and to each other in the manner shown, and the joints stopped with iron

* The Birtley Iron Company, Newcastle-on-Tyne, were the contractors for the ironwork, but a portion was supplied by the Horsely Iron Company. Mr. M'Intosh, of Bloomsbury Square, had the contract for driving the piles and fixing the work.

cement. Where the mooring-rings come, the plates are cast concave, with a hole perforated in the middle to allow a bolt to pass through, and this bolt is secured, as well as the land-ties from the main piles, to the old wharf, which was not otherwise disturbed, or to needle-piles driven adjoining it. The backing consists of a concrete of lime and gravel, in the proportion of about one to ten, extending down to the solid bottom. The coping with the water channel in its rear is of Devonshire granite; the water is conveyed from the channel at intervals by pipes, extending from gratings in the bottom in a slanting line to the lowermost plate, discharging themselves immediately above the sheet-piling.

The main piles were originally proposed to be hollow in section, according to the sketch opposite (Fig. 148); but this was given up on further con-

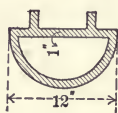


FIG. 148.—ORIGINAL FORM PROPOSED FOR BRUNSWICK.

sideration of the uncertainty of procuring sound castings of the intended form, and of the greater liability to break afterwards from a blow sidewise. The solid form shown on the plate was therefore adopted, according to which the lower lengths weighed about 28 cwt.; and that this was not too much was shown by the circumstance of several of the piles, particularly the early ones, breaking in the testing or driving, and showing in the fracture the danger of even a slight defect. The greater care subsequently taken at the foundry, and probably also greater experience in driving, made accidents of this kind of rarer occurrence in the later stages of the work; and it may be mentioned as no bad proof of the care of all parties, that of upwards of six hundred piles, including both descriptions, only sixteen broke in driving, seven being of one sort, and nine of the other: the failure was in five cases attributed to strains in driving, and to imperfections of casting in the other eleven. The sheet-piles, which bear a considerable resemblance in their general outline to those used at Downes Wharf ten years before, were proposed to be an inch thick, but it was found necessary to increase this dimension, and some of them were as much as $1\frac{1}{4}$ inches; the average, however, was not above $1\frac{1}{8}$ inches, and the weight of each pile 17 cwt. The length of the wharf is about 720 feet, and the whole weight of iron used upwards of 900 tons.

The crab engine was employed invariably, the heads of the piles being covered with a slip of $\frac{3}{4}$ -inch elm, to distribute the force of the blow equally over the iron, and prevent jarring. The monkeys used weighed from 13 to 15 cwt. each, and it was found necessary to limit the fall to a height of 3 feet 6 inches, and sometimes less, when the resistance proved more than usually great and the pile showed a tendency to turn from its straightforward course. The driving throughout was very hard, more especially at the west end, where the sheet-piles in four bays could not be forced to the full depth, the space above being in two of them made up with two plates in height, and in the other two admitting only one, instead of three as in the rest of the

work. Driving was the only means resorted to, or indeed practicable in the gravelly soil that prevailed. Had the bottom been clay or other similar substance, the plan of boring to receive the points, that has been followed elsewhere, might probably have been partially adopted in the main piles with advantage; but I should say, certainly not to the extent of depending mainly upon it for getting the piles home to their places.

I cannot quit the subject of the Brunswick Wharf without stating that his avocations alone have prevented Mr. George Lidder's association with me in the account of a work, the execution of which he had, under Messrs. Walker and Burges, the charge of superintending. Though rejoicing at the cause, I cannot help regretting the circumstance in the present instance, as such co-operation on the part of my friend would, I feel, have given this paper an interest and a value it has now but little claim to. I take this opportunity also of acknowledging my obligation to several of the gentlemen above named in connection with the previous use of iron piling, whose kindness has enabled me to make the preliminary review much fuller than I had at one time any expectation of having the power to do.

It remains for me only, in conclusion, to advert to a consideration that ought not to be lost sight of in deciding upon the eligibility of cast-iron wharfing—I mean the action of water upon it. I do not recollect any observations made so as to enable a practical inference to be drawn from them; but the importance of the subject seems to claim attention, and possibly even this notice may be the means of inducing it from those who have the opportunity.

The investigation belongs perhaps rather to chemistry than engineering, but notwithstanding the practical turn some of the most distinguished cultivators of that science have given their researches, little I believe has yet been done to explain the present question. How iron is affected by water in its various states, and in what manner the action on wrought differs from that on cast iron, are interesting points, still, so far as my information goes, to be determined; and they are not likely to be so in a satisfactory manner until some one competent to the task calls a series of well-conducted experiments in aid, as every day shows more clearly the uncertainty of analogical reasoning, however apparently strict, on such subjects. But whatever the *modus operandi* between cause and effect, that decomposition of the metal, more or less rapid, gradually goes on from the action of water, seems to admit of no doubt. Professor Faraday, in a letter to Captain Brown, says, "Cast iron is certainly liable to great injury from constant immersion in salt water, and I think you would find few, if any exceptions, provided the water and the iron are in contact."* And the saline principle, to use a somewhat antiquated form of expression, though a great accelerator of the process, does not appear to be altogether an essential to it;† at least, I know a case

* Description of a Bronze or Cast-iron Columnal Lighthouse, etc., by Capt. Brown, R.N.

† The difference between sea and other water, in operating with the galvanic

that happened in a part of the river Thames where the water cannot be said to be more than brackish at any time, and indeed is generally quite fresh, in which cast iron, after being immersed for little more than twenty years, was, on being withdrawn from the water, found so soft as to yield to the penknife; and the original surface of the iron referred to—it was the socket-plate to the heel-post of a lock-gate—had not been submitted to the tool, in which case it is well known the water would have operated with much greater effect.

But though I have thought it well to glance at the above case occurring in water, always except on rare occasions fresh, the sea is no doubt in practice the invader whose inroads are most alarming. Instances might easily be cited in proof of the ravages committed by that active enemy, though not perhaps noted so circumstantially as is desirable, but I am unwilling to lengthen this communication further, and shall therefore confine myself to a passing allusion to the example on a large scale, and after long trial, furnished by the state of the guns taken from the wreck of the *Royal George*, as described at a late meeting of the Institution;* and to a similar instance mentioned by Berzelius, in a passage which I quote at length, not so much however in confirmation of so well established a fact as the eventual decomposition of cast iron by the action of water, as for the properties mentioned of the substance into which the metal is resolved. The extract is as follows:

“Quand la fonte reste long-temps sous l'eau, elle est décomposée; l'acide carbonique contenu dans l'eau dissout le fer et l'entraîne; il reste une masse grise qui ressemble à la plombagine. Lorsqu'on retira de l'eau, il y a quelques années, les canons d'un vaisseau qui avait coulé à fond cinquante ans auparavant, aux environs de Carlsrona, on les trouva au tiers converti en une pareille masse poreuse; à peine étaient-ils à l'air depuis un quart d'heure, qu'ils commencèrent à s'échauffer tellement, que l'eau qui y restait encore s'échappa sous forme de vapeur, et qu'il fut impossible d'y toucher. Depuis, Macculloch a observé † que le corps analogue à la plombagine qui se forme ainsi présente toujours ce phénomène, et que ce corps s'échauffe presque jusqu'au rouge, en absorbant de l'oxygène. Ou ne sait pas précisément ce qui se passe dans ce cas.”—*Traité de Chimie*, Tom. III, p. 273.

battery, is much less considerable than that between the latter and distilled, but it is between salt and fresh that the practical question lies in the present case.

* *Min. of Convers.*, Vol. V, No. 12.

† The observation referred to by Berzelius in the above occurs in Macculloch's *Western Isles of Scotland* (I think in the account of the island of Mull), where an explanation of the phenomenon was first attempted, though, if on such a subject I may “hint a doubt,” not to my mind quite a satisfactory one. A more perfect solution will probably be furnished by whoever, availing himself of the powerful means of chemical analysis now possessed, may undertake such an investigation of the whole question of the action of water on iron as I have ventured to allude to in the text.

TABLE XI.—HEALD & SISCO STANDARD IRON HORIZONTAL CENTRIFUGAL PUMPS.

No.	Capacity in Gallons per Minute.	Horse-power Required for Each Foot of Lift. Minimum Quantity.	Diameter and Face of Pulley in Ins.	Floor Space Required in Inches.	Shipping Weight, Pounds.	Price of Pump, Oilers and Wrench.	Price of Pump and Primer.	No.
1½	50 to 70	.024	6×6	17×30	168	\$45	\$55	1½
1¾	75 to 100	.037	7×8	21×33	232	60	70	1¾
2	110 to 150	.054	8×8	23×37	306	75	90	2
2½	175 to 250	.086	8×8	24×38	348	90	105	2½
3	250 to 350	.124	8×8	25×39	400	110	130	3
4	450 to 600	.223	10×10	30×40	545	130	155	4
5	750 to 900	.372	15×12	34×54	826	165	195	5
6	1,000 to 1,400	.496	15×12	37×55	965	200	240	6
8	1,700 to 2,200	.844	20×12	45×63	1,500	310	375	8
10	2,200 to 3,000	1.093	24×12	51×71	2,170	395	470	10
12	3,000 to 4,000	1.40	30×14	62×78	3,050	500	...	12
15	4,800 to 6,000	2.38	40×15	77×80	7,100	850	...	15
*15	4,800 to 6,000	2.38	30×15	60×68	3,150	710	...	15
18	7,500 to 10,000	3.73	40×15	93×103	9,000	1,300	...	18
*18	7,500 to 10,000	3.73	30×16	62×70	3,500	1,150	...	18
22	12,000 to 14,000	5.96	48×20	126×130	12,000		...	22

The number of pump is also diameter of discharge opening in inches. Where more than 25 feet of discharge-pipe is attached to pump, use one of two sizes larger than pump discharge.

For No. 12 and larger sizes a foot-valve or flap-valve and ejector for priming is recommended.

TABLE XII.—LIST OF HEALD & SISCO HYDRAULIC DREDGING-AND SAND-PUMPS.

Number of Pump.	Diameter Suction and Discharge Openings, Inches.	Cubic Yards of Material they will Raise per Hour.	Horse-power Recommended for 10-foot Lift.	Diameter and Face of Pulley.	Floor Space Required, Inches.	Shipping Weight, Pounds.	Will Pass Solids, Diameter, Ins.	Price of Pump Complete, with Suction and Discharge Elbows, Flap-valve, and Ejector.	Number of Pump.
4	4	30 to 50	6	12×12	40×31	800	2	\$210	4
6	6	60 to 80	12	20×12	68×40	1,700	4½	300	6
8	8	125 to 150	22	24×14	72×48	3,400	6	475	8
10	10	200 to 300	35	30×15	94×54	4,200	8	600	10
12	12	300 to 375	45	36×20	114×66	9,000	10	850	12
15	15	400 to 500	75	42×24	154×78	12,000	10	1,450	15
18	18	500 to 700	125	48×30	160×80	13,500	10	1,900	18
20	20		...				...		20
22	22		...				...		22

* Refers to low-lift pumps.

TABLE XIII.—NUMBER OF REVOLUTIONS AT WHICH PUMPS SHOULD RUN TO RAISE WATER TO DIFFERENT HEIGHTS.

No.	5 Feet.	10 Feet.	15 Feet.	20 Feet.	25 Feet.	30 Feet.	35 Feet.	40 Feet.
1 $\frac{1}{2}$	428	604	739	854	955	1045	1131	1208
1 $\frac{3}{4}$	348	491	601	695	777	850	920	982
2	302	426	522	603	674	737	798	852
2 $\frac{1}{2}$	302	426	522	603	674	737	798	852
3	302	426	522	603	674	737	798	852
4	285	402	493	569	637	697	754	805
5	256	362	443	512	572	626	678	724
6	214	302	368	427	478	523	566	604
8	183	259	317	366	409	448	485	517
10	168	238	291	336	376	411	445	475
12	133	188	230	266	298	326	352	376
15	105	148	181	209	234	256	277	295
*15	151	213	261	301	337	369	399	426
18	105	148	181	209	234	256	277	295
*18	151	213	261	301	337	369	399	426

Above table gives *correct* speed of pumps as employed under usual conditions of pumping. If water must be forced through a number of bends and elbows, or a great length of piping, the above speed must be somewhat increased.

Use large pipes and easy bends wherever practicable, as they save power.

TABLE XIV.—TABLE OF SIZES, LIDGERWOOD SINGLE-CYLINDER, SINGLE-DRUM HOISTING-ENGINES.

Horse-power Usually Rated.	Dimen- sions of Cylinder		Weight Hoisted Single Rope, Usual Speed, Pounds.	Suitable Weight of Pile-driving Hammer for Quick Work, Lbs.	Dimensions of Hoisting-drum.			Dimensions of Bed-plate.		Dimensions of Boiler.			Estimated Shipping Weight Complete, etc., Lbs.
	Diameter, Inches.	Stroke, Inches.			Diarn. Body between Flanges, Inches.	Length Body between Flanges, Inches.	Diameter Flanges, Inches.	Width, Inches.	Length, Inches.	Diameter Shell, Inches.	Height Shell Inches.	Number of 2-inch Tubes.	
4	5	8	1200	1000	10	20	22	38	60	28	63	40	3550
6	6 $\frac{1}{4}$	8	1500	1250	10	20	22	38	60	28	60	40	3950
8	6 $\frac{1}{4}$	10	1750	1500	12	20	24	41	73	30	72	44	4850
10	7	10	2500	1800	12	20	24	41	73	32	75	48	5050
11	7	10	2500	2000	14	22	26	45	73	34	78	52	5350
12 $\frac{1}{2}$	8 $\frac{1}{4}$	10	4000	2500	14	23	29	47	73	36	75	57	6550
15	8 $\frac{1}{4}$	10	4000	2800	14	23	29	47	73	36	81	57	6750
20	8 $\frac{1}{2}$	12	6000	4000	16	26	33	54	84	40	84	80	8500
25	10	12	8000	5000	16	26	33	54	84	42	90	88	9500

* Refers to low-lift pumps.

TABLE XV.—TABLE OF SIZES, LIDGERWOOD DOUBLE-CYLINDER, DOUBLE-DRUM HOISTING-ENGINES.

Horse-power Usually Rated.	Dimensions of Cylinders.		Dimensions of Hoisting- drums.		Weight Hoisted Single Rope, Average Speed.	Suitable Weight of Pile-driving Hammer for Quick Work.	Dimensions of Boiler.			Dimensions of Bed- plate.		Estimated Ship- ping Weight with Boiler Complete.
	Diam., Inches.	Stroke, Inches.	Diam., Inches.	Length, Inches.			Diam., Inches.	Height, Inches.	Number of 2-in. Tubes.	Width, Inches.	Length Inches.	
8	5	8	12	22	2,000	1,500	32	75	48	47	80	6,500
12	6 $\frac{1}{4}$	8	14	22	3,000	2,000	36	75	57	50	86	8,000
16	6 $\frac{1}{2}$	10	14	26	4,000	2,800	38	81	68	54	89	9,000
20	7	10	14	26	5,000	3,500	40	84	80	54	89	9,550
30	8 $\frac{1}{4}$	10	14	27	8,000	5,000	42	90	88	57	94	11,400
40	8 $\frac{3}{4}$	12	16	32	10,000	8,000	50	102	124	70	117	21,000
50	10	12	16	32	12,000	10,000	53	102	150	70	117	22,000



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